6-0000 STORM DRAINAGE

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6-0000 STORM DRAINAGE

6-0100 GENERAL INFORMATION

6-0101 Drainage Systems

6-0101.1 It is the intent of § 6-0000 et seq. to require that public facilities meet or exceed applicable drainage laws.

6-0101.2 The overall drainage system is divided into 2 parts, the minor system and the major system.

6-0101.2A The minor drainage system (normally designed for the 10-yr storm) consists of storm sewer appurtenances and conduits such as inlets, manholes, street gutters, roadside ditches, swales, small underground pipe and small channels which collect the stormwater runoff and transport it to the major system.

6-0101.2B (91-06-PFM) The major system (designed for the less frequent storm up to the 100-yr level) consists of natural waterways, large man-made conduits, and large water impoundments. In addition, the major system includes some less obvious drainageways such as overland relief swales and infrequent temporary ponding at storm sewer appurtenances. The major system includes not only the trunk line system which receives the water from the minor system, but also the natural backup system which functions in case of overflow from or failure of the minor system. (See § 6-1500 et seq.)

6-0101.3 Special attention is invited to:

6-0101.3A (91-06-PFM) The current Virginia E&S Control Handbook and the Virginia Stormwater Management Handbook Volumes I & II. These handbooks address State criteria for stormwater management to be applied to control flooding and erosion.

6-0101.3B Planning Bulletin 319, "BMPs for Hydrologic Modifications," published by DEQ. The bulletin is a guide to be used whenever modifications to flowing streams are proposed.

6-0101.3C (91-06-PFM) Engineering Properties of Fairfax County Soils, published by Fairfax County Department of Public Works and Environmental Services.

6-0101.3D (91-06-PFM) Copies of the handbooks, the bulletin and the soils document are available for viewing at the Department of Public Works and Environmental Services.

6-0102 VDOT Requirements. See § 1-0602 et seq. regarding VDOT Standards.

6-0103 Metric Requirements. Until hydraulic and hydrologic design aids are available in metric units, design computations may continue to be performed in English units with the description of proposed structures converted to metric after computations are complete.

6-0200 POLICY AND REQUIREMENTS FOR ADEQUATE DRAINAGE

6-0201 Policy of Adequate Drainage

6-0201.1 In order to protect and conserve the land and water resources of this County for the use and benefit of the public, measures for the adequate drainage of surface waters shall be taken and facilities provided in connection with all land development activities. (See also § 2-602 of the Zoning Ordinance).

6-0201.2 (91-06-PFM) Adequate drainage of surface waters means the effective conveyance of storm and other surface waters through and from the development site and the discharge of such waters into a natural watercourse, i.e., a stream with a defined channel(bed and banks), or man-made drainage facility of sufficient capacity without adverse impact upon the land over which the waters are conveyed or upon the watercourse or facility into which such waters are discharged. (See § 6-0202 et seq.)

6-0201.3 (91-06-PFM) The provision of the necessary onsite and offsite easements to accomplish this also shall be required. These are to include sufficient easement extensions to property lines to permit future development reasonable access to drainageways or drainage facilities for connections.

6-0202 Minimum Requirements

6-0202.1 Determination of the size and capacity of the drainage system shall be based on the planned development, existing zoning or existing development, whichever is greater, within the watershed.

6-0202.2 The drainage system shall be designed:

6-0202.2A (91-06-PFM) To honor natural drainage divides for both concentrated and non-concentrated stormwater runoff leaving the development site. If natural drainage divides cannot be honored, each diversion from one drainage area to another may be approved by the Director in accordance with the following conditions:

6-0202.2A(1) The increase and decrease in discharge rates, volumes, and durations of concentrated and non-concentrated stormwater runoff leaving a development site due to the diverted flow shall not have an adverse impact (e,g, soil erosion; sedimentation; yard, dwelling, building, or private structure flooding; duration of ponding water; inadequate overland relief) on adjacent or downstream properties.

6-0202.2A(2) The applicant shall demonstrate to the satisfaction of the Director that the diversion is necessary to: a) improve an existing or potentially inadequate outfall condition; b) preserve a significant naturally vegetated area or save healthy, mature trees, which otherwise could not be preserved or saved, and which may be used to meet tree cover requirements instead of newly planted trees; c) maximize the water quality control and/or water quantity control provided; d) address constraints imposed by the dimensions or topography of the site to preclude adverse impacts from steep slopes and/or runoff; or e) minimize to a reasonable extent, as determined by the Director, the number of on-site stormwater management facilities.

6-0202.2A(3) The construction or grading plan shall include a written justification for the proposed diversion and a detailed analysis of both concentrated and non-concentrated stormwater runoff leaving a development site for each affected downstream drainage system in accordance with the requirements of § 6-0203. The extent of downstream analysis shall be performed to a point where the diverted flow is returned to its natural course. However, the analysis for a non-bonded lot grading plan proposing a diversion of less than 0.5 CFS for the 10-year design storm may be terminated at a point that satisfies § 6-0203.2, if that point is upstream of the point where the diverted flow is returned to its natural course. Otherwise, the extent of downstream review shall be performed to a point where the diverted flow is returned to its natural course and in accordance with § 6-0203, and whichever point results in the furthest downstream review shall govern.

6-0202.2A(4) A diversion shall not be approved if it adversely impacts the adequacy of downstream drainage systems; creates new floodplain areas on adjacent or downstream properties; alters Resource Protection Area boundaries; aggravates or creates a non-compliance with provisions governing elevations and proximity to 100-year water surface elevations; changes the drainage area at points where perennial streams begin; or changes the total drainage area of a watershed depicted on the County map of Watersheds, as may be amended.

6-0202.2B To account for both off-site and on-site surface waters.

6-0202.2C (91-06-PFM) To convey such waters to a natural water course at the natural elevation, or an existing storm drainage facility. (See § 6-201.2.)

6-0202.2D (91-06-PFM) To discharge the surface waters into a natural watercourse or into an existing or proposed man-made drainage facility of adequate capacity except as may be provided for in § 6-0203.

6-0202.3 (91-06-PFM) Concentrated stormwater runoff leaving a development site shall be discharged directly into an adequate natural or man-made receiving channel, pipe or storm sewer system or the developer must provide a drainage system satisfactory to the Director to preclude an adverse impact (e.g. soil erosion; sedimentation; yard flooding; duration of ponding water; inadequate overland relief) on downstream properties and receiving channels in accordance with § 6-0203, as well as a proportional improvement of the predevelopment conditions (§ 6-0203.4 and § 6-0203.5). If the developer chooses to install a storm drainage system, the system shall be designed in accordance with established, applicable criteria for such systems.

6-0202.4 (91-06-PFM) Concentrated stormwater runoff leaving a development site shall not aggravate or create a condition where an existing dwelling or a building constructed under an approved building permit floods from storms less than or equal to the 100-year storm event. If such a dwelling or building exists, detention for the 100-year storm event shall be provided in accordance with § 6-0203.5.

6-0202.5 (91-06-PFM) Concentrated surface waters shall not be discharged on adjacent or downstream property, unless an easement expressly authorizing such discharge has been granted by the owner of the affected land or unless the discharge is into a natural watercourse, or other appropriate discharge point as set forth above.

6-0202.6 (91-06-PFM) The owner or developer may continue to discharge stormwater which has not been concentrated (i.e. sheet flow) into a lower lying property if:

6-0202.6A (91-06-PFM) The peak rate after development does not exceed the predevelopment peak rate; or

6-0202.6B(1) (91-06-PFM) The increase in peak rate or volume caused by the development will not have any adverse impact (e.g. soil erosion, sedimentation, duration of ponding water, inadequate overland relief) on the lower lying property as determined by the Director; and

6-0202.6B(2) (91-06-PFM) The increase in peak rate or volume caused by the development will not aggravate any existing drainage problem or cause a new drainage problem on the downstream property.

6-0202.7 (91-06-PFM) Increases in peak rates or volumes of sheet flow that may cause any adverse impact on lower lying properties shall be discharged into an adequate existing drainage system or the developer shall provide an adequate drainage system satisfactory to the Director to preclude any adverse impact upon the adjacent or downstream property.

6-0202.8 (91-06-PFM, 31-90-PFM) Drainage structures shall be constructed in such a manner that they may be maintained at a reasonable cost. To facilitate design, construction, and maintenance, drainage facilities shall meet and conform, insofar as practical, to County and VDOT standards. However, small private drainage systems may be acceptable (See § 6-0205) for solving drainage problems that may develop during the course of construction of a new development or for implementation by property owners in existing developments. See § 6-0205 and Plate 1-6 (1M-6) for construction details and example.

6-0202.9 (91-06-PFM) If off-site downstream construction and easements are necessary, no plans shall be approved until such storm drainage easements have been obtained and recorded. If the downstream owner or owners refuse to give or to sell such easements, the owner or developer may request condemnation of the easements by the County at the developer's cost. If the County declines to institute condemnation, the plan shall not be approved.

6-0202.10 (91-06-PFM) Storm sewers shall be discharged into the area least likely to erode.

6-0202.10A Generally, it is better to discharge at the floodplain limit into an adequate watercourse channel leading to the main streambed, rather than disturb the floodplain by extending the storm sewer.

6-0202.10B If an adequate watercourse channel does not exist, the only alternative is to discharge into the main streambed.

6-0202.10C In either case, energy dissipation devices are required.

6-0202.11 The requirements of Chapter 104, (Erosion and Sedimentation Control) of the Code, and the further requirements for protection of streambeds by detention-retention of surface waters, set forth in § 6-0000 et seq. must be satisfied. Additionally, BMP requirements to protect water quality must be met, if applicable (§ 6-0400 et seq.).

6-0202.12 (91-06-PFM) The on-site major storm drainage system must be designed in accordance with § 6-1500 et seq.

6-0202.13 (27-89-PFM) Consideration must be given in the preparation of the plans to preclude adverse impacts due to higher rates and volumes of flow that will occur during construction. Special consideration shall be given to the design of sediment traps which discharge into existing residential yards. In this case, in order to reduce concentrated flows and simulate existing sheet flow conditions, the 10-yr peak discharge shall be designed to be not greater than 0.5 CFS (0.014 CMS) using a minimum runoff C factor of 0.6 for all areas to be disturbed.

6-0202.14 In those cases in which the drainage plans of a proposed development do not satisfy these minimum requirements because necessary off-site facilities or improvements are lacking, the developer shall delay development until the necessary off-site facilities or improvements are constructed or other arrangements, satisfactory to the Director, are made.

6-0203 (91-06-PFM) Analysis of Downstream Drainage System

6-0203.1 The downstream drainage system shall be analyzed to demonstrate the adequacy of the system (§ 6-0203.3), or it shall be shown that there is no adverse impact to the downstream system as well as a proportional improvement of the predevelopment conditions (§ 6-0203.4 and § 6-0203.5)

6-0203.2 The extent of the review of the downstream drainage system shall be:

6-0203.2A To a point that is at least 150 ft (46 meters) downstream of a point where the receiving pipe or channel is joined by another that has a drainage area that is at least 90% of the size of the first drainage area at the point of confluence; or

6-0203.2B To a point at which the total drainage area is at least 100 times greater than the contributing drainage area of the development site; or

6-0203.2C To a point that is at least 150 ft (46 meters) downstream of a point where the drainage area is 360 acres (1.46 km^2) or greater.

6-0203.2D When using §§ 6-0203.2A and 6-0203.2C for the extent of review, the analysis must be to a point where all the cross-sections are adequate in the farthest downstream reach of 150 feet . A minimum of three cross-sections shall be provided in the 150-foot reach. If the detention method described in §6-0203.4C is used, the three cross-sections in the farthest downstream reach of 150 feet shall be limited to showing a defined channel or a man-made drainage facility and checking for flooding as described in § 6-0203.4C(3) and § 6-0203.5.

6-0203.2E The Director may require analysis farther downstream when the submitted narrative described in § 6-0204 and all related plats and plans are insufficient to show the true impact of the development on surrounding and other lower lying properties, or if there are known drainage problems downstream.[1]

6-0203.2F Cross-section selection and information shall be determined in accordance with Chapter 5 of the latest edition of the Virginia Erosion and Sediment Control Handbook (Virginia Department of Conservation and Recreation) under the section titled "Determination of Adequate Channel." Cross-sections shall be shown on the plans with equal horizontal and vertical scales.

6-0203.2G If the downstream owner(s) refuse to give permission to access the property for the collection of data, the developer shall provide evidence of this refusal and make arrangements satisfactory to the Director to provide an alternative method for the collection of data to complete the outfall analysis (e.g., through the use of photos, aerial surveys, "as built" plans, County topographic maps, soils maps, and any other relevant information).

6-0203.3 Adequacy of all natural watercourses, channels and pipes shall be verified as follows:

6-0203.3A The developer shall demonstrate that the total drainage area to the point of analysis within the channel is 100 times greater than the contributing drainage area of the development site; or

6-0203.3B(1) Natural watercourses shall be analyzed by the use of a 2-year frequency storm to verify that stormwater will not overtop channel banks nor cause erosion of channel bed or banks;

6-0203.3B(2) All previously constructed man-made channels shall be analyzed by the use of a 10-year frequency storm to verify that stormwater will not overtop channel banks and by the use of a 2-year frequency storm to demonstrate that stormwater will not cause erosion of channel bed or banks;

6-0203.3B(3) Pipes, storm sewer systems and culverts, which are not maintained by VDOT, shall be analyzed by the use of a 10-year frequency storm to verify that stormwater will be contained within the pipe, system, or culvert; and

[1] These drainage problems may be documented as parts of County watershed or drainage studies, complaints on file with the County, or complaints on file at the offices of County Supervisors.

6-0203.3B(4) Pipes, storm sewer systems and culverts, which are maintained by VDOT, shall be analyzed by the use of the 10-year or greater frequency storm in accordance with VDOT requirements.

6-0203.3C Determinations of the adequacy of drainage systems shall be performed in accordance with methods contained in Chapter 5 of the latest edition of the Virginia Erosion and Sediment Control Handbook (Virginia Department of Conservation and Recreation) under the section titled “Determination of Adequate Channel.”

6-0203.4 A proportional improvement and no adverse impact to the downstream drainage system shall be shown by one of the following methods:

6-0203.4A Critical Shear Stress Method

6-0203.4A(1) If the outfall is inadequate due to erosive velocities along the extent of review, which is described in § 6-0203.2, the critical shear stress method may be used to show no adverse impact due to erosive velocities. The erosive work on the channel for the post-development conditions shall be reduced to a level below the erosive work on the channel under pre-development conditions by the required proportional improvement. The required proportional improvement of the downstream system at each inadequate cross-section is the ratio of the post-development C times A (see § 6-0803 for a description of C times A) for the contributing drainage area of the site to the existing development C times A for the entire drainage area at that cross-section. The required proportional improvement is computed as follows:

$$P_i = [C_d\ A_d / C_{cs}\ A_{cs}] \text{ X } 100 \text{ where,}$$

P_i = Required Proportional Improvement (%)
C_d = Runoff Coefficient for the Contributing Drainage Area of the Site in a Post-development Condition
A_d= Contributing Drainage Area of the Site
C_{cs} = Runoff Coefficient for the Contributing Drainage Area to the Cross-section in a Existing Development Condition
A_{cs}= Contributing Drainage Area to the Cross-section

Each inadequate cross-section along the extent of review shall then be analyzed for the following:

6-0203.4A(2) The shear stress for both the predevelopment condition and the post-development condition for the 2-year storm shall be plotted in relation to time at each cross-section. On each graph, the permissible shear stress also shall be plotted. The permissible shear stress is based on the soil type, and may be determined for cohesive soils from Plate 76-6 (Plate 76M-6) and for non-cohesive soils from Plate 77-6 (Plate 77-M-6). The soil type may be determined by field test or the soil type designated on the County soils maps may be used. If the soil type is designated using the County soils maps, the most conservative permissible shear stress for the soil type shall be used. The plans shall indicate how the soil type was determined. The area between the permissible shear stress and the actual shear stress on the graph is erosive work on the channel. The erosive work for the post-development condition shall be less than the erosive work for predevelopment condition by a percentage equal to the required proportional improvement.

The shear stress on the channel can be calculated using the following formula:

$$t = gRS \text{ where,}$$

t = shear stress in lb/sq.ft. (N/m^2)
g = unit weight of water is 62.4 lb/ft^3 (9810 N/m^3)
R = hydraulic radius in ft (m)
S = slope of the channel bed

6-0203.4B Channel Capacity Method

6-0203.4B(1) If the outfall is inadequate due to inadequate capacity along the extent of review, which is described in § 6-0203.2, the channel capacity method may be used to show no adverse impact due to overtopping. The largest storm that does not exceed the actual channel, pipe, or culvert capacity under pre-development conditions shall be determined for the cross-section that is most frequently over its capacity. The post-development peak flows for the above storm and the 2-year and 10-year storms shall be reduced to a level below the pre-development conditions by a percent equal to the required proportional improvement. See § 6-0203.4A(1) for a description of the required proportional improvement.

6-0203.4C Detention Method [2]

6-0203.4C(1) It shall be presumed that no adverse impact and a proportional improvement will occur if on-site detention is provided as follows and the outfall is discharging into a defined channel or man-made drainage facility:

6-0203.4C(1)(i) Extended detention of the 1-year storm volume for a minimum of 24 hours. If extended detention of the BMP volume (see § 6-0400 et seq.) also is provided, the 24 hours shall be applied to the difference between the 1-year storm volume and the BMP volume; and

6-0203.4C(1)(ii) In order to compensate for the increase in runoff volume, the 2-year and 10-year post-development peak rates of runoff from the development site shall be reduced below the respective peak rates of runoff for the site in good forested condition (e.g., for NRCS method, a cover type of "woods" and a hydrologic condition of "good"). This reduction results in a proportional improvement and is computed as follows:

$$R_i = [1 - (V_f / V_d)] \text{ X } 100 \text{ where,}$$

R_i = Reduction of Peak Flow Below a Good Forested Condition (%)
V_f = Runoff Volume from the Site in a Good Forested Condition
V_d= Runoff Volume from the Site in a Post-Developed Condition

The calculation of the cumulative volumes shall be based on the NRCS (formerly SCS) methodology described in § 6-0802 or other methods as approved by the Director.

6-0203.4C(1)(iii) Computations demonstrating the 1½-year post-development peak rate of runoff from the development site does not exceed the 1½-year peak rate of runoff for the site in good forested condition are optional. The 1½- year storm is used to obtain Leadership in Energy and Environmental Design (LEED) certification.

6-0203.4C(2) If this method is used, each outfall from the site shall be analyzed independently and the allowable release rate shall be based on the area of the site that drains to the outfall under predevelopment conditions.

6-0203.4C(3) If this method is used, the downstream review analysis shall be limited to providing cross-sections to show a defined channel or man-made drainage facility, and checking for flooding of existing dwellings or buildings constructed under an approved building permit from the 100-year storm event for the extent of review described in § 6-0203.2A, B , C and D.

6-0203.4D Other scientifically valid methods, which show no adverse impact regarding erosion or capacity for an inadequate outfall and show proportional improvement, may be approved by the Director.

6-0203.5 In accordance with § 6-0202.4, if an existing dwelling or a building constructed under an approved building permit, which is located within the extent of review described in § 6-0203.2, is flooded by the 100-year storm, the peak flow of the 100-year storm at the development site shall be reduced to a level below the pre-development condition by a percent equal to the required proportional improvement. See § 6-0203.4A(1) for a description of the required proportional improvement.

6-0204 (91-06-PFM) Submission of Narrative Description

6-0204.1 In addition to plats, plans, and other documents that may be required, a description of each outfall of the storm drainage system from the development site shall be submitted as part of the relevant subdivision construction plan or site plan and shall include the following:

6-0204.1A The additional submission shall include a narrative and sketches describing the major elements (pipe, channel, natural watercourse stream, etc.) of each outfall drainage system, including any discharges of non-concentrated surface waters from the development site. Photographs may also be included to assist in the description of the outfall.

[2] Because of the long detention times resulting from this method, consideration shall be given to hydrology, soils and extended detention when choosing the appropriate landscaping for the detention facility.

6-0204.1B Downstream Review

The downstream review, divided into reaches, as required by § 6-0203, shall:

6-0204.1B(1) Note the existing surrounding topography, soil types, embankments, vegetation, structures, abutting properties, etc., which may be impacted by drainage;

6-0204.1B(2) In cases where the developer seeks to establish that the existing downstream facilities and/or natural waterways are adequate to receive the drainage from the development site, provide sufficient cross-section information, associated graphs, and computations to support the assertion of adequacy, in accordance with § 6-0203.3;

6-0204.1B(3) In cases where the downstream facilities are inadequate and the developer proposes to use the detention method, in accordance with § 6-0203.4C, provide sufficient information to (i) establish the existence of a defined channel or man-made drainage facility to receive the concentrated discharge from the development site, and (ii) demonstrate at least the minimum required proportional improvement, as described in § 6-0203.4C(1), will be achieved;

6-0204.1B(4) In cases where the downstream facilities are inadequate and the developer proposes to use the critical shear stress or channel capacity method, in accordance with § 6-0203.4A and § 6-0203.4B, provide sufficient cross-sections, associated graphs, and computations to demonstrate (i) there will be no adverse impacts and (ii) at least the minimum required proportional improvement, as described in § 6-0203.4A(1), will be achieved;

6-0204.1B(5) Provide sufficient information to demonstrate that (i) there will be no flooding of existing dwellings, or buildings constructed under an approved building permit, by the 100-year storm event, or (ii) any existing flooding condition will not be aggravated by drainage from the development site and a proportional improvement is made in accordance with § 6-0203.5; and

6-0204.1B(6) Include a written opinion, certified, signed, and sealed by the submitting professional, that (i) the requirement of adequacy of the downstream drainage system(s) is met or the development will meet the no adverse impact condition and achieve the required proportional improvement of predevelopment conditions; (ii) if any portion of the outfall drainage system is a natural watercourse, the cross-sections analyzed and included on the plan are representative of stream reaches for the entire extent of review for the natural watercourse portion of the system; and (iii) there will be no flooding of existing downstream dwellings, or buildings constructed under an approved building permit, by the 100-year storm event, or that any existing flooding condition will not be aggravated by drainage from the development site.

6-205 Small Private Drainage System (See Plate 1-6 (1M-6) (31-90-PFM)

6-0205.1 The intended uses for these small private drainage systems are generally meant to apply exclusively to solving existing drainage problems that may develop during the course of construction of a new development or for implementation by property owners in existing developments. They are not to be used in the design of new developments to circumvent the normal requirements for a standard public drainage system. Accordingly, they are not intended to convey large flows from major swales or drainage areas. That is, design flows will typically be in the range of 1 to 3 CFS (0.03 to 0.09 CMS).

6-0205.2 If the system is located on more than 1 private property, private easements in favor of the other system owner(s) must be mutually granted in order to ensure proper operation and maintenance of the system. In addition, when the system is located on more than 1 private property, a County construction permit will be required and as a part of that permit's requirements, a maintenance/hold harmless agreement, which will run with the land ownership, will need to be executed by the system owners and recorded in the land records of the County. Maintenance of these systems will be the responsibility of the system owner(s), not of the County.

6-0205.3 Extreme caution should be exercised in locating the terminal discharge point of the system so that downstream property owners will not be adversely impacted. Riprap (small rock) should be used to dissipate the discharge energy and reduce the discharge velocity to a non-erosive rate. Where connection to a County drainage system is proposed, DPWES, Maintenance and Stormwater Management Division, is to be contacted for permission.

6-0205.4 (31-90-PFM) Example: A homeowner has excessive runoff thru his backyard. The estimated size of the watershed is 0.5 acres. The estimated percent impervious cover is 60%.

Step 1: Determine the amount of design runoff in CFS. Use Table 6.1 as a guide:

TABLE 6.1 HYDROLOGY – SMALL PRIVATE DRAINAGE SYSTEM (31-90-PFM)

Size of Watershed Draining to Point of Interest Acres (ha)	Estimate the % of Impervious Cover in the Watershed (e.g., roofs, pavement, sidewalks)			
	20%	40%	60%	80% or more
	Low Density Residential CFS (CMS)	Medium Density Residential CFS (CMS)	High Density Residential CFS (CMS)	Commercial Industrial CFS (CMS)
0.25 (0.1)	.85 (0.024)	1.10 (0.031)	1.40 (0.040)	1.90 (0.054)
0.50 (0.2)	1.70 (0.048)	2.30 (0.065)	2.90 (0.082)	3.90 (0.011)
1.00 (0.4)	3.40 (0.096)	Public System Required	Public System Required	Public System Required

From Table 6.1 the Design Flow is estimated at 2.9 CFS or say 3 CFS.
Step 2: Determine the size of the pipe and details of the inlet structure. Use Table 6.2 as a guide.

TABLE 6.2 HYDRAULICS – SMALL PRIVATE DRAINAGE SYSTEM

Pipe Dia. Inches (mm)	(A.) Pipe Grade (G)				(B.) Available Ponding Headwater (HW)			
	.5%	1.0%	2.0%	4.0%	1' (0.3M)	2' (0.6M)	3' (0.9M)	4' (1.2M)
	CFS (CMS)	CFS (CMS)	CFS (CMS)	CFS (CMS)	CFS (CMS)	CFS (CMS)	CFS (CMS)	CFS (CMS)
8 (200)	1.0 (0.028)	1.4 (0.040)	2.0 (0.057)	2.8 (0.079)	1.69 (0.048)	2.39 (0.068)	2.93 (0.083)	3.37 (0.095)
10 (250)	1.8 (0.051)	2.6 (0.074)	3.6 (0.102)	5.2 (0.147)	2.52 (0.071)	3.56 (0.101)	4.36 (0.123)	6.65 (0.188)

Given: The runoff to the point of interest is 3 CFS. The elevation at the ground equals 200.0'. The elevation at the outfall 100' away equals 196.0' at the watercourse (i.e., stream invert).

Solution: (1) Use a 2.25 ft² grate at 0.5' ponding depth (capacity is 3 CFS at 0.5'). Try 10" diameter pipe at 2% (from Table 6.2). 2% at 100' equals 2' of drop in elevation for grade. Therefore, 196' (invert elevation at water course) + 2' (elevation needed for grade) = 198' elevation of pipe invert at drop inlet. (2) Check ponding depth capacity: 200' – 198' – 0.42' to centerline of pipe = 1.58'. Interpolating Table 6.2B gives 3.1 > 3.0 required; therefore OK.

Results: The grate top elevation equals 199.5. 10" diameter PVC rigid non-perforated pipe; invert elevation out of drop inlet equals 198.0'; invert elevation at watercourse 100' away equals 196.0'; install "Y" or brick structure with solid ¼" thick steel plate on top along pipe at 50' intervals for clean-out access; install 3' long, 18" wide riprap (4" to 8" diameter stone) at terminus of pipe.

6-0300 POLICY ON DETENTION OF STORMWATERS

6-0301 General Policy

6-0301.1 It is the intent of this policy to encourage the use of various methods for the on-site detention of stormwater in the interest of minimizing the adverse effects of increased stormwater runoff (resulting from development of land within the County) on all downstream drainageways.

6-0301.2 (31-90-PFM) It also is the intent of this policy to encourage a regional approach in the implementation of stormwater detention, rather than numerous small, less effective individual on-site ponds.

6-0301.3 Detention facilities must be provided in all storm drainage plans proposed for development in the County submitted for review and approval unless waived by the Director.

6-0301.4 (35-91-PFM) Regional dry ponds or extended dry ponds are the preferred types of stormwater management facility, except in locations where the County's Regional Stormwater Management Plan calls for a wet BMP pond. The use of wet ponds in residential developments is restricted to regional facilities or to residential developments where there are no other reasonable options available for compliance with the water quality requirements.

6-0301.5 (46-94-PFM) A wet pond is a regional wet pond if it is approved as such as a part of the County's regional stormwater management plan. In addition, a wet pond may be deemed by the County to be a regional wet pond if it (1) is the functional equivalent of a regional wet pond or (2) has an upstream watershed area of 100 acres (40 ha) or more, and a detention capacity and BMP capacity capable of serving the entire upstream watershed.

6-0302 Detention Measures

6-0302.1 Except where otherwise prohibited, detention, either alone or in combination with other measures, is an acceptable option for meeting the County and State requirement for protecting receiving waterways from erosion and flooding resulting from (developed) runoff.

6-0302.2 On-site detention of stormwater is desirable in many cases to alleviate existing downstream drainage problems and to preclude the development of new ones.

6-0302.2A Detention is mandatory where the existing downstream drainage system is clearly inadequate and its expansion or improvement is either financially prohibitive or unacceptable for aesthetic or other compelling reasons.

6-0302.2B In some areas of a watershed, detention may cause increased peak flows to occur on the major streams and tributaries. Therefore, the downstream impact must be carefully investigated.

6-0302.2C The Director may prohibit detention of stormwater where and when it is not in the best interests of the County.

6-0302.3 The release rate from ponding areas shall approximate that of the site prior to the proposed development for the design storm, but adequate alternate drainage must be provided to accommodate major storm flows.

6-0302.4 The rooftops of buildings may be used for detention, but care should be taken to design the buildings to accommodate the additional live loading involved if the depth exceeds 3" (75mm).

6-0302.5 Detention pools or basins in parks (subject to the approval of FCPA), playing fields, parking lots or storage areas can be constructed to reduce peak runoff downstream by providing on-site storage.

6-0302.5A Care must be taken to ensure that such ponds do not become nuisances or health hazards.

6-0302.5B The design engineer should strive to design detention facilities which require minimal maintenance. The maintenance responsibility shall be clearly stated on the plans.

6-0302.5C Where dual purpose facilities are provided, flat grades encountered, or poor draining soils found, provisions for adequate low flow drainage may be required.

6-0302.6 Porous material may be used where practical as an alternative to conventional impervious parking area paving.

6-0302.6A This material would allow the stormwater to be absorbed more readily by the ground rather than adding to additional runoff.

6-0302.6B This practice is not applicable to areas where a high water table exists or where subsoil conditions are not suitable.

6-0302.6C Design engineers are encouraged to investigate and propose experimental uses of new or existing products and methods including porous asphalt pavement where such use may appear appropriate.

6-0302.6D Parking areas surfaced with gravel or rock must be approved by the Director, in accordance with Paragraph 9 of § 11-202 of the Zoning Ordinance, or § 7-0504 et seq.

6-0303 Location of Detention Facilities

6-0303.1 (32-90-PFM) All non-regional "wet ponds" (ponds with a permanent water surface) in residentially zoned areas must be maintained by the homeowners association and a private maintenance agreement must be executed before the construction plan is approved. Dry detention ponds and regional wet detention ponds, including those constructed to serve BMP facilities, located in residentially zoned areas, including condominium developments, shall be within County storm drainage easements, and shall be maintained by DPWES.

6-0303.2 (46-94-PFM) Detention and BMP facilities located in industrial, commercial, institutional, apartment developments and rental townhouses must be maintained by the property owner, and a Private Maintenance Agreement must be executed before the construction plan is approved.

6-0303.3 (38-93-PFM) Retention, detention and/or BMP facilities may not be located in RPAs unless an exception is approved under provisions of Chapter 118 (Chesapeake Bay Preservation Ordinance) of the Code.

6-0303.4 Although this policy is primarily concerned with maintaining post-development peak outflow at the level of the pre-development condition, it may be applied under certain conditions for the purpose of correcting an existing inadequate outfall. When used in this fashion, such a facility also may aid in meeting the requirement for adequate drainage.

6-0303.5 When new development lies in the Occoquan watershed, the provisions of § 6-0400 et seq. relating to BMPs also apply.

6-0303.6 (46-94-PFM) Wherever stormwater management facilities are planned in areas within 300' (90m) of a residence or active recreational area, the design shall be directed specially toward the safety aspects of the facility and shall conform to the requirements of § 6-1606; including such features as mild bottom slopes along the periphery of a detention pond extending out to a point where the depth exceeds 2' (0.6m), flat lateral and longitudinal slopes where concrete low flow channels are used, outlet structures with properly fastened trash racks which will inhibit unauthorized entrance, and posted warning signs.

6-0303.7 In addition, credit for recreational open space shall not be allowed in those areas where detention facilities are located unless the area can reasonably be used for recreational purposes. For example, some detention ponds could be used for active recreational use if the low flows are totally separated from the play areas by a piping system.

6-0303.8(24-88-PFM, 83-04-PFM) Underground detention facilities may not be used in residential developments, including rental townhouses, condominiums and apartments, unless specifically waived by the Board of Supervisors (Board) in conjunction with the approval of a rezoning, proffered condition amendment, special exception, or special exception amendment. In addition, after receiving input from the Director regarding a request by the property owner(s) to use underground detention in a residential development, the Board may grant a waiver if an application for rezoning, proffered condition amendment, special exception, and special exception amendment was approved prior to, June 8, 2004, and if an underground detention facility was a feature shown on an approved proffered development plan or on an approved special exception plat. Any decision by the Board to grant a waiver shall take into consideration possible impacts on public safety, the environment, and the burden placed on prospective owners for maintenance of the facilities. Any property owner(s) seeking a waiver shall provide for adequate funding for maintenance of the facilities where deemed appropriate by the Board. Underground detention facilities approved for use in residential developments by the Board shall be privately maintained, shall be disclosed as part of the chain of title to all future homeowners (e.g. individual members of a homeowners or condominium association) responsible for maintenance of the facilities, shall not be located in a County storm drainage easement, and a private maintenance agreement in a form acceptable to the Director must be executed before the construction plan is approved. Underground detention facilities may be used in commercial and industrial developments where private maintenance agreements are executed and the facilities are not located in a County storm drainage easement.

6-0303.9 (35-91-PFM) Detention or structural BMP facilities, including 10-yr flood storage areas associated with such facilities, shall not be located on individual buildable single family attached and detached residential lots, or any part thereof for the purpose of satisfying the detention or BMP requirements of the Subdivision Ordinance or Zoning Ordinance. However, detention and BMP facilities may be constructed on individual lots to satisfy the detention and BMP requirements for each lot. County maintenance for detention and BMP facilities on such individual lots will not be provided.

6-0400 STORMWATER RUNOFF QUALITY CONTROL CRITERIA (38-93-PFM)

6-0401 General Information and Regulations

6-0401.1 The Board has established a WSPOD in the Occoquan Watershed to prevent water quality degradation of the Occoquan Reservoir due to pollutant loadings within the watershed. WSPOD boundaries have been established on the Official Zoning Map. Use limitations are established which require that there shall be water quality control measures designed to reduce the projected phosphorus runoff by at least 1/2 for any subdivision or use requiring site plan approval unless a modification or waiver is approved by the Director.

6-0401.2 The Board has established Chesapeake Bay Preservation Areas (CBPAs) consisting of RPAs and RMAs throughout the entire County to protect the quality of water in the Chesapeake Bay and its tributaries (Chapter 118 of the Code). RPA and RMA components are identified in § 118-1-7 of the Code. Performance criteria have been established which require that there shall be water quality control measures designed to prevent a net increase in nonpoint source pollution from new development based on average land cover conditions and to achieve a 10% reduction in nonpoint source pollution from redevelopment. For purposes of § 6-0400 et seq., the average land cover condition is 18% imperviousness. These criteria are implemented as follows:

6-0401.2A For new development, the projected total phosphorus runoff pollution load for the proposed development shall be reduced by no less than 40% compared to phosphorus loads projected for the development without BMPs. This requirement shall not apply to any development that does not require a site plan pursuant to Article 17 of the Zoning Ordinance, that does not require subdivision approval pursuant to the Subdivision Ordinance, and that does not result in impervious cover of 18% or greater on the lot or parcel on which the development will occur.

6-401.2B (79-03-PFM) For redevelopment of any property not currently served by one or more BMPs, the required reduction in phosphorus loads will be computed for each site based on the following formula:

$[1-0.9(\text{"I"}_{pre}/\text{"I"}_{post})] \times 100 = \%\ \text{P removal}$

where "I"_{pre} is the predevelopment percent impervious area and "I"_{post} is the postdevelopment percent impervious area.

6-0401.2C For redevelopment of any property that is currently and adequately served by one or more BMPs, the projected phosphorus runoff pollution load after redevelopment shall not exceed the existing phosphorus runoff pollution load.

6-0401.2D "Redevelopment" means the substantial alteration, rehabilitation, or rebuilding of a property for residential, commercial, industrial, or other purposes where there is no net increase in impervious area by the proposed redevelopment within an RPA and no more than a net increase in impervious area within an RMA of 20% relative to conditions prior to redevelopment, or any construction, rehabilitation, rebuilding, or substantial alteration of residential, commercial, industrial, institutional, recreational, transportation, or utility uses, facilities or structures within an Intensely Developed Area.

6-0401.2E (79-03-PFM) These requirements apply to any development or redevelopment in a CBPA unless a waiver or modification is approved by the Director. Waivers or modifications shall be subject to the following criteria:

6-0401.2E(1) (79-03-PFM) The requested waiver or modification is the minimum necessary to afford relief;

6-0401.2E(2) (79-03-PFM) Granting the waiver or modification will not confer upon the applicant any special privileges that are denied by Chapter 118 of the County Code to other property owners who are subject to its provisions and are similarly situated;

6-0401.2E(3) (79-03-PFM) The waiver or modification is in harmony with the purpose and intent of Chapter 118 of the County Code and is not of substantial detriment to water quality;

6-0401.2E(4) (79-03-PFM) The waiver or modification request is not based upon conditions or circumstances that are self-created or self-imposed;

6-0401.2E(5) (79-03-PFM) Reasonable and appropriate conditions are imposed, as warranted, that will prevent the activity from causing a degradation of water quality; and

6-0401.2E(6) (79-03-PFM) Other findings, as appropriate and required by Chapter 118 of the County Code, are met.

6-0401.3 The Board has also adopted stormwater runoff quality control requirements with certain approved rezoning and special exception applications.

6-0401.4 The water quality control measures described in § 6-0000 et seq. are called BMPs. The term BMP refers to a practice, or combination of practices, which has been determined by the Director to be the most effective practicable means of preventing or reducing the amount of pollution generated by nonpoint sources to a level compatible with water quality goals.

6-0402 Stormwater Quality Control Practices. The BMP policy where required is incorporated into the stormwater management program in the following manner:

6-0402.1 The Director may require the control of offsite areas draining to proposed BMPs which would not operate at the listed phosphorus removal efficiency, because of hydraulic overloading, if these areas were left uncontrolled. Control of offsite areas for this purpose will not be required in excess of an amount which would be considered equivalent to 100% site coverage.

6-0402.2 (56-96-PFM) Guidance for the design of BMPs can be found in Chapter 4 of the Northern Virginia BMP Handbook (NVPDC/ESI, 1993). Such manual may be modified by the Board to apply specifically to Fairfax County. Guidance on the design of BMPs specifically intended for use with small area grading plans is available from DPWES.

6-0402.3 (56-96-PFM) For purposes of § 6-0400 et seq., the following standard BMPs, sizing rules, and their associated phosphorus removal efficiencies, based on available water quality planning studies, are accepted:

TABLE 6.3 PHOSPHORUS REMOVAL EFFICIENCIES (56-96-PFM)

BMP	Sizing Rule	Phosphorus[1] Removal (%)
Extended Detention[2] Dry Pond (48-hr)	Plate 2-6 (2M-6)	40

Wet Pond[3] Design 1 Design 2	$2.5 \times V_r$ + extended detention $4.0 \times V_r$	45 50
Infiltration[4] Design 1 Design 2 Design 3	0.5 in./impervious acre (32mm/impervious ha) 1.0 in./impervious acre (64mm/impervious ha) 2-yr, 2-hr storm	50 65 70
Natural Open Space[5]	N/A	100
Regional Ponds[6] Dry Pond Wet Pond	Plate 2-6 (2M-6) $4.0 \times V_r$	50 65
Sand Filter[7]	0.5 in./impervious acre (32mm/impervious ha)	60
Pervious Pavement[8] Design 1 Design 2	Captures and treats a water quality volume of 0.5 in (1.27 cm) without infiltration with infiltration Captures and treats a water quality volume of 1.0 in. (2.54 cm) without infiltration with infiltration	 35 50 40 65
Bioretention Basin/Filter	0.5 in./impervious acre (32mm/impervious ha) 1.0 in./impervious acre (64mm/impervious ha)	50 65
Vegetated Swale	Volume Based Design 0.5 in./impervious acre (32mm/impervious ha) 1.0 in./impervious acre (64mm/impervious ha) Flow Based Design Hydraulic residence time of 18 minutes	 50 65 30
Tree Box Filter	0.5 in./impervious acre (32mm/impervious ha) 1.0 in./impervious acre (64mm/impervious ha)	50 65

Vegetated Roof[8]	N/A	40
Reforestation[9]	N/A	70

[1]Phosphorus (as total P), the limiting nutrient for algal productivity in local receiving waters, is used as an indicator of water quality. Measures that control phosphorus also will control many other pollutants.

[2]A minimum drawdown time of 48 hr is required for the BMP storage volume.

[3]V_r is the volume of runoff from the mean storm. It is computed based on an average annual rainfall of 40" (1016mm) per year and an average of 100 storms per year multiplied by the rational formula "C" factor. Design 1 incorporates extended detention above the permanent pool equal to the Plate 2-6 (2M-6) value.

[4]Infiltration may be used only on soils designated by a professional authorized by the State to provide such information as adequate for the purpose. Special attention must be given to construction and maintenance practices for infiltration.

[5]For purposes of BMP efficiencies, "open space" in residential areas is defined as perpetually undisturbed Homeowners Association (or "common") areas placed in floodplain or conservation easements and without other encumbrances. Full credit for utility easements equal to or less than 25' (7.6m) in width and which meet the above criteria is allowed. The Director may allow "open space" credit for undisturbed areas in utility easements greater than 25' (7.6m) in width on a case by case basis. Any areas located within private lots or with maintained landscaping or active recreational areas are not to be included in "open space" determinations. In nonresidential areas, "open space" is defined as perpetually undisturbed areas placed in floodplain or conservation easements and without other encumbrances. Credit for utility easements equal to or less than 25' (7.6m) in width and which meet the above criteria is allowed. The Director may allow "open space" credit for undisturbed areas in utility easements greater than 25' (7.6m) in width on a case by case basis. Open space used for BMP credit which is not already in a floodplain easement shall be placed in a recorded conservation easement with metes and bounds which shall also be shown on the plat. BMP credit for open space, which is dedicated to the County during the land development process, may be assigned to the remaining portions of the original site on approval by the Board.

[6]Regional ponds are those facilities which are part of the regional stormwater management plan adopted by the Board or substitutes and additions to the plan approved by DPWES. All ponds for which regional BMP credit is requested must be approved by DPWES. Regional ponds generally control a watershed of 100 acres (40 ha) or more in size. However, the Director may allow regional BMP credit for smaller ponds constructed to satisfy requirements of the regional plan.

[7]Sand filters shall be privately owned and maintained.

[8]In applying the computational procedure in Chapter 4 of the Northern Virginia BMP Handbook to demonstrate compliance with the phosphorus removal requirement for the site, the “C” factor for pervious pavements and vegetated roofs should be set at 0.9 to correctly credit the phosphorus removal provided by these controls. This will result in a different weighted “C” factor than that used to compute stormwater runoff.

[9]In applying the computational procedure in Chapter 4 of the Northern Virginia BMP Handbook to demonstrate compliance with the phosphorus removal requirement for the site, the “C” factor ratio for reforestation should be set at 1.0 because reforestation is being treated as a land use credit rather than a structural control.

6-0402.4 Other innovative BMP measures may be permitted but, due to the design variables that could affect their appropriateness and efficiencies, percentages are not listed above. A request for use of these techniques will be reviewed on a case by case basis and approved by the Director as appropriate. The developer must provide full details and supporting data including:

- Justification
- Technical details with research data supporting efficiencies
- Maintenance considerations and program (private maintenance will generally be required for innovative BMP facilities)
- Any safety considerations
- Aesthetic considerations
- Location and interaction with populated areas
- Pest control program, if required.

6-0402.5 The efficiencies set forth in § 6-0402.6 apply only to the portion of the site served by each practice; however, credit may be allowed for control of runoff pollution from off-site areas. Additional credit is not allowed for practices in series without the Director's approval.

6-0402.6 The following options will be considered to comply with the stormwater runoff quality control criteria in RPAs and RMAs:

6-0402.6A (79-03-PFM) Incorporation on the site of BMPs that achieve the required control as set forth in § 6-0401. For the purposes of this subsection, the "site" may include multiple projects or properties that are adjacent to one another or lie within the same drainage area where a single BMP or a system of BMPs will be utilized by those projects in common to satisfy water quality protection requirements;

6-0402.6B (79-03-PFM) Compliance with a locally adopted regional stormwater management program which may include a Virginia Pollution Discharge Elimination System (VPDES) permit issued by the Department of Environmental Quality to a local government for its municipally owned separate storm sewer system discharges, that is reviewed and found by the Chesapeake Bay Local Assistance Board to achieve water quality protection equivalent to that required by this Article (BMP credit for a pro rata share payment to the regional stormwater management program will be computed based on the portion of the site which drains to an existing or proposed regional pond providing water quality control.); or,

6-0402.6C (79-03-PFM) Compliance with a site-specific VPDES permit issued by the Department of Environmental Quality, provided that the local government specifically determines that the permit requires measures that collectively achieve water quality protection equivalent to that required by this Article.

6-0402.7 Developers, in coordination with DPWES, are strongly encouraged to seek cooperation with other planned developments in their watershed area in order to construct combined facilities which could serve several developing sites. This regional approach to stormwater management would result in facilities that are not only efficient in terms of stormwater quality control, but are also cost effective and land saving.

6-0402.8 The following information is required on all site and subdivision plans to show compliance with the water quality control requirements of § 6-0000 et. seq.

6-0402.8A A brief narrative summarizing how water quality control requirements are being provided for the site.

6-0402.8B A map showing all subareas used in the computations of weighted average "C" factors, BMP storage, and phosphorus removal including offsite areas, open space, and uncontrolled areas.

6-0402.8C Open space used for BMP credit should be delineated on the plan sheets with the note "Water quality management area. BMP credit allowed for open space. No use or disturbance of this area is permitted without the express written permission of the Director of the Department of Public Works and Environmental Services."

6-0402.8D Computations used to determine BMP outflow rates and size outlet structures.

6-0402.8E Computations of BMP facility storage requirements.

6-0402.8F Computations of BMP phosphorus removal for the site.

6-0402.8G Statement of maintenance responsibility for the BMPs (public or private).

Additional information may be required by the Director to justify use of nonstandard designs or in unusual situations.

6-0500 POLICY ON OFF-SITE DRAINAGE IMPROVEMENTS

6-0501 Purpose and Intent. In the interest of public health, safety and welfare when the appropriate land use has been determined for any area to be developed, the Director may require the developer to show that off-site downstream drainage can be accommodated (considering the planned development of the contributing watershed) without damage to existing facilities or properties before such development is approved for construction.

6-0502 General Policy (36-92-PFM)

6-0502.1 The County's pro rata share program for off-site drainage improvements involves assessing new development for a proportionate share of the cost of off-site drainage improvements. It provides the County a funding source for the portion of the cost of drainage improvements necessitated by the increased runoff from new development. The County may require pro rata share contributions for off-site storm drainage improvements in all areas where pro rata share improvements have been planned as part of the general drainage improvement program.

6-0502.2 Pro rata share payments will not be reduced based on a development providing normal on-site detention/BMP requirements. Pending the availability of pro rata monies, developer costs for off-site construction of drainage improvements or implementation of a regional detention pond may be considered for a pro rata share assessment reduction and/or reimbursement. Developer reimbursement will be facilitated only by written agreement executed with the Board prior to construction plan approval. The developer's maximum amount of a pro rata share assessment reduction and/or reimbursement will be limited to the developer costs which are over and above the normal costs that would be incurred in developing the property. The maximum amount of annual pro rata share reimbursement to a developer would be established in the reimbursement agreement. Generally, the annual reimbursement to any individual developer would be based on the relationship of the developer's excess costs to the total costs of all improvements required in the watershed coupled with the actual amount of pro rata monies collected in any given year. Pro rata share reimbursements will start after completion of the drainage improvements by the developer and acceptance of the improvements by the County. The reimbursements will continue for a maximum of 15 yr pursuant to the written agreement.

6-0600 POLICY ON PROPORTIONATE COST OF OFF-SITE DRAINAGE IMPROVEMENTS

6-0601 General Requirements

6-0601.1 (36-92-PFM) Development within a watershed involving a change of land use therein normally results in an increase in impervious areas resulting in a greater quantity as well as a more rapid and frequent concentration of stormwater runoff and the discharge of pollutants associated with the development.

6-0601.2 (36-92-PFM) The construction of storm drainage improvements is required along waterways as watershed development progresses to alleviate flood damage, arrest deterioration of existing drainageways and minimize environmental damage to the downstream receiving waters.

6-0601.3 The extent and character of such improvements shall be designed to provide for the adequate correction of deficiencies.

6-0601.4 Improvements shall extend downstream to a point where damages to existing properties ascribable to the additional runoff are minimized.

6-0601.5 The purpose and intent is to require a developer of land to pay his pro rata share of the cost of providing reasonable and necessary drainage facilities located outside the property limits of the land owned or controlled by the developer, but necessitated or required, at least in part, by the construction or improvement of his subdivision or development.

6-0602 Pro Rata Share Studies (36-92-PFM)

6-0602.1 When directed to do so by the County Executive, the Director of DPWES or his designee shall study and compute the total estimated cost of the general improvement program projects required to serve the watershed when and if such watershed is fully developed in accordance with the adopted comprehensive plan for the watershed or the current zoning of the land, whichever is higher.

6-0602.2 The total estimated cost of the drainage improvement program shall include design, land acquisition, utility relocation, construction, and administrative costs for the projects contained in the improvement program.

6-0602.3 The computation of total estimated costs shall include any engineering study for the watershed or improvement program.

6-0602.4 When this cost is computed it shall be updated every 6 months by applying the Engineering News Record Construction Cost index value to the construction costs.

6-0602.5 The above study with its attendant cost figures shall constitute the general improvement program for the affected watershed.

6-0603 General Drainage Improvement Program. (36-92-PFM) When a general drainage improvement program has been established, a pro rata share of the total cost of the program shall be determined as follows:

6-0603.1 The County shall determine the estimated increased volume and velocity of stormwater runoff, expressed as an increase in impervious area, for the watershed when fully developed in accordance with the adopted comprehensive plan or the current zoning of the land, whichever is higher.

6-0603.2 The total cost of the drainage improvement program for the watershed divided by the increase in impervious area for the watershed shall be computed by the County to determine the pro rata share rate for that watershed.

6-0603.3 The developer shall determine the increase in impervious area for the development. If the development is located within more than 1 major watershed, then the developer shall determine the increase in impervious area for each portion of the development which lies within each major watershed. The major watersheds are defined on the County's 1"=4000' (1:48000) Watersheds map. A specific site must be divided into drainage areas conforming to the major watershed boundaries. The total site area within each major watershed must be included in the computation whether it is controlled by a storm sewer, detention/retention runoff facility, BMP, or sheet runoff design. Pro rata share reduction will not be allowed for normal on-site detention/BMP requirements.

6-0603.4 The developer shall provide the computations showing the increase in impervious area for the development to the County as part of subdivision plan, site plan, public improvement plan and development plan submittal requirements. The County will compute the developer's pro rata share assessment by multiplying the respective watershed rate by the development's increase in impervious area. The assessment rates are available in the Environmental and Facilities Inspection Division, DPWES.

6-0604 Pro Rata Share Payments (36-92-PFM)

6-0604.1 The payment of the pro rata share assessment shall be due prior to subdivision, site plan, or public improvement plan approval.

6-0604.2 When development occurs in a subdivision which has been previously approved and no pro rata share paid, or where a landowner is improving an existing lot which results in an increase in impervious area, the payment of the pro rata share shall be made before the issuance of any building permits, in accordance with State and County Codes.

6-0604.3 The pro rata share assessments received shall be kept in separate accounts for each of the watershed improvement programs until such time as they are expended for the watershed improvement program.

6-0604.4 Payments received shall be expended only for the established watershed improvement program for which the payment was calculated. Any interest that accrues on such payments shall accrue to the benefit of the County.

6-0604.5 All storm drainage pro rata share accounts existing as of October 1, 1992 were abolished by transferring assets into separate funds for the support of each separate respective watershed improvement program. After the transfer of such assets, depositors who had met the terms of any pro rata share agreements prior to July 1, 1990 received any outstanding interest which had accrued up to the date of transfer, and were released from any further obligation under those existing agreements. All transferred assets became the sole property of the County.

6-0700 POLICY ON WHAT MAY BE DONE IN FLOODPLAINS

6-0701 Applicability

6-0701.1 In the interpretation of Part 9 of Article 2 of the Zoning Ordinance and in recognition of the County's desire to participate in the National Flood Insurance Program, it is the intent of § 6-0000 et seq. that the following goals be met:

6-0701.1A The preservation of the hydraulic and flood carrying capacity within the altered or relocated portion of the natural channel of any adopted floodplain;

6-0701.1B The preservation of the storage characteristics of floodplains, and

6-0701.1C The preservation of the natural environment.

6-0701.2 Therefore, some improvements which will accommodate the increased runoff from changes or improvements within the watershed without unacceptably elevating floodplain or stream levels may be needed within floodplains, streams and/or drainageways, particularly within improved or developed areas.

6-0701.3 The improvements may take the form of piping or channelization with concrete, riprap, or gabions; streambed clearing; removal of obstructions; reduction of constrictions; stabilization of stream bottoms and/or banks to eliminate or reduce erosion; widening, deepening or realigning of streams. The objective of such improvements is to provide the necessary hydraulic characteristics to accommodate the anticipated stormwater flow without damaging adjacent properties.

6-0702 Alteration of Floodplains

6-0702.1 Where there is a major alteration or relocation of the natural channel of an adopted floodplain, the Federal Insurance Administrator, the DEQ and affected adjacent political jurisdictions shall be notified.

6-0702.2 Improvements shall include the removal of silt and debris which may clog or damage downstream drainage structures or property and the filling or draining of ponding areas and stagnant pools which are potential vermin shelters and mosquito breeding areas.

6-0702.3 The decision to perform any of the above must be in conformance with the Zoning Ordinance, Chapter 118 (Chesapeake Bay Preservation Ordinance) of the Code, and with PL 92-500, Section 404 Permit Program as administered by the COE.

6-0702.3A The intent of the Federal law is to "…insure that the chemical/biological integrity of waters of the United States is protected…".

6-0702.3B Section 404 applies to the bed and banks of navigable waterways as defined in the Federal Register, Volume 40, Number 144, Part IV, dated July 25, 1975; and to the adjacent wetlands, effective in June 1977.

6-0703 Use Regulations in Floodplain Areas (86-04-PFM)

6-0703.1 All newly proposed subdivision lots located in or adjacent to a floodplain must contain sufficient area of land above the 100-yr floodplain to allow a residence to be constructed thereon, taking into consideration the minimum yard requirements of the Zoning Ordinance.

6-0703.2 No part of any building lot in a cluster subdivision may extend into a floodplain, except as provided in Part 6 of Article 9 of the Zoning Ordinance for cluster subdivisions in the R-C, R-E and R-1 Districts and cluster subdivisions in the R-3 and R-4 Districts which have a minimum district size of two (2) acres but less than three and one-half (3.5) acres, and Part 9 of Article 2 of the Zoning Ordinance and § 101-2-8 of the Code for cluster subdivisions in the R-2 District and cluster subdivisions in the R-3 and R-4 Districts which have a minimum district size of three and one-half (3.5) acres or greater. No clearing or grading in the floodplain shall be permitted, except as provided for in Parts 6 and 9 of Article 2 of the Zoning Ordinance.

6-0703.3 The lowest part of the lowest floor of any such residence must be at least 18" (457.2mm) above the 100-yr flood level.

6-0704 Floodplain Development Standards. All development permitted in the floodplain area shall, at a minimum, comply with all applicable Federal and State laws and the following standards, except that the Director may impose more restrictive standards which may be warranted by the specific conditions.

6-0704.1 The developer must provide factual information that any proposed structure will not adversely affect the existing 100-yr flood level; and that adequate emergency access is available to the structure during periods of maximum flooding. The applicant must specify the 100-yr water surface elevation(s) on the plan.

6-0704.2 The lowest part of the lowest floor level of any proposed residential structure must be located at least 18" (457.2mm) above the 100-yr water surface elevation and a minimum horizontal distance of 15' (4.6m) must be provided between the 100-yr water surface and the structure proper.

6-0704.3 Non-residential structures, or parts thereof, where permitted, may be constructed below the regulatory flood elevation provided that these structures are designed to preclude and/or withstand inundation to an elevation of at least the regulatory (100-yr) flood elevation. The submitting engineer or architect shall specify the elevation and certify that the structure has been floodproofed, and that the elevation and flood-proofing comply with applicable Federal and State requirements.
6-0704.4 Compensatory excavation normally will be required for fills unless waived for environmental reasons.

6-0705 Warning and Disclaimer of Liability

6-0705.1 The degree of flood protection required by the PFM is considered reasonable for regulatory purposes. Larger floods may occur on rare occasions or flood heights may be increased by man-made or natural causes, such as bridge openings restricted by debris.

6-0705.2 Therefore, § 6-0000 et seq. does not imply that areas outside the floodplain areas, or land uses permitted within such areas, will be free from flooding or flood damages under all conditions.

6-0705.3 Additionally, the grant of a permit or approval of a site, subdivision or land development plan in an identified floodplain area or flood hazard area shall not constitute a representation, guarantee, or warranty of any kind by any official or employee of the County of the practicability or safety of the proposed use, and shall create no liability upon the County, its officials or employees.

6-0705.4 In the event that the Director issues a permit under the provisions of the PFM, the applicant may be asked to execute an agreement holding the County harmless from the effects caused by the construction or existence of the permitted use. Such an agreement shall be recorded among the land records of the County.

6-0800 HYDROLOGIC DESIGN

6-0801 Acceptable Hydrologies (27-89-PFM)

TABLE 6.5 ACCEPTABLE HYDROLOGIES – APPLICATIONS

Name of Hydrology	200 Acres (81 ha) and Under	Over 200 Acres (81 ha)	Retention/Detention Facilities
SCS*	X	X	X

Rational Formula	X	O	X***
Anderson Formula	O	X	O
Other**	X	X	X

X = Acceptable hydrology
O = Unacceptable hydrology
* = Recommended hydrology
** = With approval of the Director
*** = Watersheds less than 20 acres (8ha) only, provided that the "C" factor for the unimproved areas does not exceed 0.15 on the 2-yr storm.

6-0802 SCS Hydrology. (27-89-PFM) SCS Hydrology consists of Technical Release Number 20 (TR-20) and Technical Release Number 55 (TR-55) including the COE HEC-1 software, SCS applications. This hydrology is preferred and acceptable for all applications.

6-0803 Rational Formula. (47-95-PFM) The Rational Formula, $Q = C_fCIA$, is acceptable for drainage areas of 200 acres (81 ha) and under, except it is not authorized for designing detention/retention facilities greater than 20 acres (8 ha). The Rational Formula may be used for the design of detention/retention facilities of 20 acres (8 ha) and less provided that the "C" factor for unimproved areas does not exceed 0.15 on the 2-yr storm and the facility is in full compliance with all other requirements of § 6-1600 et seq.

Q = Rate of runoff in CFS
C_f = Correction Factor for ground saturation
C = Runoff Coefficient (ratio of runoff to rainfall)
I = Rainfall Intensity in in./hr
A = Area of drainage basin in acres

<u>C_f Values</u>

1.0 - 10-yr or less
1.1 - 25-yr
1.2 - 50-yr
1.25 - 100-yr

6-0803.1 Runoff Coefficient (C) used to compute flow to the point of interest shall be the composite of the "C" factors for all the areas tributary to the point of interest. Table 6.6 gives the runoff coefficients to be used for the different zoning classifications. For cluster areas and when clay soil is encountered the higher values of "C" shall be used.

6-0803.2 (27-89-PFM) Rainfall Intensity (I) shall be determined from the rainfall frequency curves shown in Plate 3-6 (3M-6) or Table 6.7 (for incremental unit hydrograph). The 10-yr storm frequency shall be used to design the storm drains (minor drainage systems); the 100-yr storm frequency shall be used to design the drainageways of the major drainage system.

6-0803.3 Time of Concentration (t_c) is the sum of the inlet time plus the time of flow in the conduits from the most remote inlet to the point under consideration. Flow time in conduits may be estimated by the hydraulic properties of the conduit. Inlet time is the time required for the runoff to reach the inlet of the storm sewer and includes overland flow time and flow time through established surface drainage channels such as swales, ditches and street gutters.

6-0803.3A Recommended inlet times are also shown in Table 6.6.

6-0803.3B Storm drainage systems may be designed based on zoning classification or type of surface. In general, when designing drainage facilities based on type of surface, the runoff coefficient for each inlet is selected as follows:

$$C = \frac{A_1C_1 + A_2C_2 + \ldots + A_nC_n}{A_1 + A_2 \ldots + A_n}$$

$A_1, A_2 \ldots A_n$ = Areas of different surfaces
$C_1, C_2 \ldots C_n$ = Runoff coefficients for different types of surface

Select inlet time from Table 6.6 based on C value.

6-0803.3C If an inlet time must be estimated, the following are suggestions to assist the designer:

6-0803.3C(1) Estimate the overload flow time, time for runoff to reach established surface drainage channels such as street gutters and ditches. Plate 4-6 (4M-6) can be used for overland flow.

6-0803.3C(2) Estimate the time of flow through the established surface drainage channels from the channel's hydraulic properties. Plate 5-6 (5M-6) can be used for streets and parking lots that have curb and gutter. The Mannings Equation or methods described in § 6-1000 et seq. can be used for swales and ditches.

6-0803.3C(3) Specific procedures of estimating inlet times for use with SCS hydrology are provided in the SCS TR-55 manual, Urban Hydrology for Small Watersheds.

6-0803.3D Judgement should be used in estimating time of concentration or any portion of time of concentration. Often the initial inlet time may be based on the first few inlet areas.

6-0803.3D(1) If the uppermost area has low runoff rates with long times of concentration (such as parks and cemeteries) and major portions of the lower area have high runoff rates with short times of concentration, then the first inlet time may not necessarily be based solely on its own land use.

6-0803.3D(2) The above statements also would be true of the converse case; that is, the uppermost area producing high runoff rates with short times of concentration and the lower areas producing low runoff rates with long time of concentration.

6-0803.4 Area (A). Areas shall be determined from field run topography, current USGS quadrangle sheets, or County Topographical Maps. Watershed maps showing applicable divides, contributing areas and adopted Comprehensive Plan recommendations or existing zoning, whichever is greater, must accompany all computations.

6-0804 Anderson Formula. (27-89-PFM) The Anderson Formula, $Q = 230\ KRA^{(x)}\ T^{(-0.48)}$, may be used for rates of runoff for areas greater than 200 acres (81ha), except it shall not be used for designing detention/retention facilities.

Caution: This method was developed for use in Northern Virginia and Southern Maryland and should not be used in other areas.

Q = Rate of runoff in CFS
K = Coefficient of imperviousness
R = Flood frequency ratio
A = Drainage basin area in $mi.^2$
x = Area exponent
T = Lag time in hours

6-0804.1 Coefficient of Imperviousness (K) is obtained by K = 1.00 + 0.015(I) where I is the percentage of basin area covered with impervious surface. The percentage impervious may be computed or may be taken from Table 6.6. When the drainage basin consists of different percentages of imperviousness then the average percent imperviousness, I Average, shall be calculated as follows:

$$I_{Avg} = \frac{A_1 I_1 + A_2 I_2 + \ldots + A_n I_n}{A_1 + A_2 + \ldots + A_n}$$

$A_1, A_2 \ldots A_n$ = Areas of different percentages of impervious
$I_1, I_2 \ldots I_n$ = Percent imperviousness for each area

6-0804.2 Flood Frequency Ratio (R). For a given storm recurrence interval and percent imperviousness, the flood frequency ratio R, is obtained from Plate 6-6 (6M-6).

6-0804.3 Area (A). Areas shall be determined by the latest topographic information. Generally, current USGS quadrangle sheets will be adequate.

6-0804.4 Area Exponent (^{x})

^{x} = 1.0 for areas greater than 200 acres but less than 1 $mi.^2$.
^{x} = 0.82 for areas 1 $mi.^2$ and greater.

6-0804.5 Lag Time (T) = $Y\ (L/S^{1/2})^z$

L = Distance in miles along the primary water point of interest to the drainage basin boundary.

S = S is an index of basin slope. It is determined as the average slope, in ft/mi., of the main watercourse between points located 10% and 85% of the length, L, upstream from the point of interest.

Y, ^{z} = The Y coefficient and ^{z} exponent are shown in Plate 7-6 (7M-6).

After computing the length-slope ratio, the lag time, T, may be determined using Plate 7-6 (7M-6).

6-0804.5A The top line shall be used for natural drainage basins, basins with fewer or no storm sewers.

6-0804.5B The middle line shall be used for developed drainage basins, basins where the tributaries are sewered and the main channels are natural and/or rough lined (rubble or grass).

6-0804.5C The bottom line shall be used for completely sewered and developed basins having smooth lined (concrete, brick or metal) main channels.

6-0804.5D The lag time line or equation used shall be based on the planned ultimate development of the drainage basin and main channels.

6-0804.6 Example using Anderson Formula:

6-0804.6A Given: Area of drainage basin = 1300 acres
Planned development = school
Length of drainage basin = 1.30 mi.
Elevation at 10% (0.13 mi.) upstream from point of interest = 170.0'
Elevation at 85% (1.10 mi.) upstream from point of interest = 300.0'
Planned drainage: Tributaries will be sewered and main channels will remain natural or grass-lined.

6-0804.6B Design Storm: 10-yr frequency storm

$Q = 230\ KRA^{(x)}\ T^{(-0.48)}$

6-0804.6B(1) K (coefficient of imperviousness) = 1.000 + 0.015 I (%). From Table 6.6, for school development, the percent imperviousness I = 50%.

K = 1.000 + 0.015 (50) = 1.75

6-0804.6B(2) R (flood – frequency ratio). From Plate 6-6 (6M-6), for 10-yr recurrence interval and 50% imperviousness, R = 1.7.

6-0804.6B(3) A (area) = 1300 acres x 43,560 ft^2/acre x $(1\ mi./5{,}280')^2 = 2.03\ mi.^2$.

6-0804.6B(4) $^x = 0.82$ for areas larger than 1 $mi.^2$.

6-0804.6B(5) T (lag time) = $Y(L/S^{1/2})^Z$

L = length of drainage basin (mi.) from point of interest to upper boundary
L = 1.30 mi.

S (index of basin slope) ft/mi.

$$= \frac{(E1.@85\%L)-(E1.@10\%L)}{(75\%L)}$$

$$S = \frac{300-170}{(.75)(1.30)} = \frac{130'}{.975mi.} = 134\ ft/mi.$$

From Plate 7-6 (7M-6): Y=0.9 and $^Z = 0.50$

$T = 0.9\ (1.30/134^{1/2})^{0.50}$

T = 0.302

$Q = 230\ KRA^{(x)}\ T^{(-0.48)}$

$= (230)(1.75)(1.70)(2.03)^{0.82}/(0.302)^{0.48}$

= 1222.82/0.563

Q = 2,172 CFS

6-0805 Other Hydrologies. (27-89-PFM) It is recognized that there are many hydrologies available, especially in the form of computer software. Other hydrologies may be approved by the Director for specific applications provided it is demonstrated that the alternatives are appropriate for the purpose intended.

6-0805.1 The lowest range of runoff coefficients may be used for flat areas (areas where the majority of the grades are 2% and less).
6-0805.2 The average range of runoff coefficients should be used for intermediate areas (areas where the majority of the grades are from 2% to 5%).

6-0805.3 The highest range of runoff coefficients shall be used for steep areas (areas where the majority of the grades are greater than 5%), for cluster areas, and for development in clay soils areas.

TABLE 6.6 RUNOFF COEFFICIENTS AND INLET TIMES (27-89-PFM)

ZONING CLASSIFICATION	Runoff Coefficients	% Impervious	Inlet Times (minutes)
Business, Commercial & Industrial	0.80 – 0.90	90	5
Apartments & Townhouses	0.65 – 0.75	75	5-10
Schools & Churches	0.50 – 0.60	50	10 – 15*
Single Family Units			
Lots 10,000 ft^2 (929 m^2)	0.40 – 0.50	35	
Lots 12,000 ft^2 (1115 m^2)	0.40 – 0.45	30	
Lots 17,000 ft^2 (1579 m^2)	0.35 – 0.45	25	
Lots ½ acre (2023 m^2) or more	0.30 – 0.40	20	
Parks, Cemeteries and Unimproved Areas **	0.25 – 0.35	15	To be Computed
TYPE OF SURFACE			
Pavements & Roofs	0.90	100	According to zoning classification of composite runoff coefficient
Lawns	0.25-0.35	0	
Open Water[3,4]	0.9	0	
Reforested Areas[2]	0.25-0.35	0	
Vegetated Roofs[4] Extensive Systems Intensive Systems	 0.50 0.40	N/A[5]	
Pervious Pavement[4] Porous Asphalt Pavement Permeable Pavement Blocks	 (I-1.1)/I (I-3.0)/I I=peak rainfall intensity	N/A[5]	

	(in/hr)		

1) However, for design of yard inlets i.e., locations and throat capacities, in residential areas, drainage computations shall use a 5-minute time of concentration, or alternatively, a site specific calculation to justify usage of a longer time of concentration. Computations for design of pipes may continue to use the 10- to 15-minute time of concentration.
2) For unimproved areas containing less than 5% impervious cover and storm frequencies 2-yr or less, use C = 0.10 to 0.20.
3) The runoff coefficient for open water areas such as lakes and streams is set at 0.9 because all rainfall falling on open water is converted directly to runoff. For unimproved areas containing less than a total of 5% open water plus impervious cover, the open water areas may be ignored in computing composite runoff coefficients.
4) Composite runoff coefficients for drainage areas that include significant areas of open water, pervious pavements, or vegetated roofs should not be computed directly from the percentage of impervious area. Use the weighted average of the runoff coefficients to compute the runoff.
5) Values for percent imperviousness have not been assigned to pervious pavement and green roofs. For hydrologic purposes, they respond as pervious or partially pervious surfaces. In determining land use for application of Chesapeake Bay Preservation Ordinance development/redevelopment criteria, they are treated as impervious surfaces.

6-0806 INCREMENTAL UNIT HYDROGRAPH – 1 IMPERVIOUS ACRE

TABLE 6.7 INCREMENTAL UNIT HYDROGRAPH INTENSITIES-INCHES/HOUR

TIME (Minute)	t_c=5 Minute				t_c=10 Minute				t_c=15 Minute			
	2-YR	10-YR	25-YR	100-YR	2-YR	10-YR	25-YR	100-YR	2-YR	10-YR	25-YR	100-YR
5	5.45	7.27	8.27	9.84	2.57	3.25	3.42	3.68	1.65	2.20	2.44	2.81
10	3.51	4.68	5.34	6.37	4.60	5.92	6.77	8.10	3.18	4.24	5.92	5.99
15	2.60	3.46	3.95	4.73	3.40	4.53	5.29	6.47	3.90	5.10	5.86	7.05
20	2.08	2.77	3.15	3.74	2.36	3.14	3.65	4.44	3.27	4.36	4.88	5.69
25	1.72	2.29	2.62	3.13	1.82	2.43	2.85	3.50	2.31	3.08	3.40	3.89
30	1.46	1.94	2.23	2.65	1.49	1.99	2.33	2.86	1.76	2.34	2.66	3.17
35	1.28	1.68	1.93	2.33	1.25	1.67	2.97	2.43	1.42	1.89	2.22	2.73
40	1.10	1.47	1.70	2.07	1.06	1.41	1.71	2.17	1.17	1.56	1.89	2.40
45	1.00	1.31	1.53	1.88	0.91	1.21	1.49	1.93	0.97	1.29	1.63	2.16
50	0.89	1.18	1.38	1.69	0.78	1.04	1.33	1.78	0.80	1.07	1.42	1.98
55	0.82	1.08	1.26	1.55	0.69	0.92	1.21	1.67	0.67	0.89	1.26	1.83
60	0.74	0.99	1.16	1.42	0.60	0.80	1.10	1.58	0.55	0.73	1.10	1.68
65	0.68	0.91	1.06	1.30	0.55	0.73	1.01	1.45	0.50	0.67	1.01	1.54
70	0.62	0.83	0.97	1.18	0.50	0.67	0.92	1.32	0.46	0.61	0.92	1.40
75	0.56	0.74	0.87	1.07	0.45	0.60	0.83	1.19	0.41	0.55	0.83	1.26
80	0.49	0.66	0.77	0.95	0.40	0.53	0.73	1.05	0.37	0.49	0.73	1.12
85	0.43	0.58	0.68	0.83	0.35	0.47	0.64	0.92	0.32	0.43	0.64	0.98
90	0.37	0.50	0.58	0.71	0.30	0.40	0.55	0.79	0.28	0.37	0.55	0.84
95	0.31	0.41	0.48	0.59	0.25	0.33	0.46	0.66	0.23	0.30	0.46	0.70
100	0.25	0.33	0.39	0.47	0.20	0.27	0.37	0.53	0.18	0.24	0.37	0.56
105	0.19	0.25	0.29	0.36	0.15	0.20	0.28	0.40	0.14	0.18	0.28	0.42
110	0.12	0.17	0.19	0.24	0.10	0.13	0.18	0.26	0.09	0.12	0.18	0.28
115	0.06	0.08	0.10	0.12	0.05	0.07	0.09	0.13	0.05	0.06	0.09	0.14
120	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

TABLE 6.7 (Cont'd) INCREMENTAL UNIT HYDROGRAPH INTENSITIES-INCHES/HOUR

TIME (Minute)	t_c=20 Minute				t_c=25 Minute				t_c=30 Minute			
	2-YR	10-YR	25-YR	100-YR	2-YR	10-YR	25-YR	100-YR	2-YR	10-YR	25-YR	100-YR
5	1.49	1.98	1.77	1.43	0.96	1.28	1.16	0.98	0.60	0.80	0.87	0.97
10	2.53	3.37	3.37	3.36	1.80	2.40	2.35	2.26	1.18	1.57	1.69	1.88
15	3.15	4.20	4.64	5.33	2.44	3.25	3.46	3.79	1.74	2.32	2.51	2.80
20	3.42	4.56	5.25	6.32	2.87	3.83	4.31	5.05	2.25	3.00	3.31	3.79
25	3.12	4.16	4.55	5.15	3.02	4.03	4.70	5.75	2.64	3.52	3.99	4.73
30	2.27	3.02	3.32	3.78	2.92	3.89	4.39	5.17	2.76	3.71	4.30	5.22
35	1.67	2.22	2.54	3.03	2.51	3.35	3.60	3.99	2.61	3.48	3.99	4.78
40	1.37	1.83	2.11	2.55	2.01	2.68	2.77	2.90	2.27	3.03	3.38	3.92
45	1.19	1.58	1.83	2.23	1.54	2.05	2.14	2.28	1.87	2.49	2.70	3.04
50	1.06	1.41	1.64	2.00	1.19	1.58	1.73	1.96	1.48	1.97	2.18	2.52
55	0.95	1.27	1.50	1.87	0.97	1.29	1.48	1.77	1.19	1.58	1.82	2.20
60	0.88	1.17	1.40	1.75	0.84	1.12	1.33	1.65	0.99	1.32	1.57	1.97
65	0.81	1.07	1.28	1.60	0.77	1.03	1.22	1.51	0.91	1.21	1.44	1.81
70	0.73	0.98	1.17	1.46	0.70	0.93	1.11	1.38	0.83	1.10	1.31	1.64
75	0.66	0.88	1.05	1.31	0.63	0.84	1.00	1.24	0.74	0.99	1.18	1.48
80	0.59	0.78	0.93	1.17	0.56	0.75	0.89	1.10	0.66	0.88	1.05	1.31
85	0.51	0.68	0.82	1.02	0.49	0.65	0.78	0.96	0.58	0.77	0.92	1.15
90	0.44	0.59	0.70	0.88	0.42	0.56	0.67	0.83	0.50	0.66	0.79	0.99
95	0.37	0.49	0.58	0.73	0.35	0.47	0.55	0.69	0.41	0.55	0.65	0.82
100	0.29	0.39	0.47	0.58	0.28	0.37	0.44	0.55	0.33	0.44	0.52	0.66
105	0.22	0.29	0.35	0.44	0.21	0.28	0.33	0.41	0.25	0.33	0.39	0.49
110	0.15	0.20	0.23	0.29	0.14	0.19	0.22	0.28	0.17	0.22	0.26	0.33
115	0.07	0.10	0.12	0.15	0.07	0.09	0.11	0.14	0.08	0.11	0.13	0.16
120	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

6-0900 CLOSED CONDUIT SYSTEM

6-0901 Design Flow. (88-05-PFM) The closed conduit system shall be designed for a 10-yr rainfall frequency when its intended use is to function as the minor drainage system. If the system is in the VDOT right-of-way, it should be designed to have no surcharge during the 10-year design storm. Systems in the VDOT right-of-way should be designed for the 50-year storm and checked for the 100-year design storm to prevent flooding of underpasses or other depressed roadways where ponded water can only be removed through the storm sewer system. Design flows shall be determined by methods discussed in § 6-0800 et seq., and pipes will be sized by the amount of runoff actually entering the system.

6-0902 Storm Sewer Pipe

6-0902.1 Size of storm sewer pipe may be determined by the Manning Formula which is expressed as:

$Q = VA = 1.49/n \times r^{2/3} \times S^{1/2} A$
$(Q = VA= 1/n \times r^{2/3} \times S^{1/2} A)$

Q = Quantity of flow in CFS (CMS)
V = Velocity of flow in FPS (MPS)
A = Required area in ft^2 (m^2)
n = Coefficient of roughness
r = Hydraulic radius in ft (m)
$r = \dfrac{\text{cross-sectional area of flow}}{\text{wetted perimeter}}$
S = Slope of energy gradient in ft/ft (m/m).
(See § 6-1005)

6-0902.2 Adjustments of pipe sizes as determined by the Manning Formula may be necessary due to hydraulic gradient considerations. The Manning Formula is shown in nomograph form on Plate 8-6 (8M-6). Other guidelines related to size and configuration of storm sewer pipe are as follows:

6-0902.2A (88-05-PFM) Minimum size of pipe to be used outside of the VDOT right-of-way will be 12" (300mm) diameter where the distance between access openings is 50' (15m) or less and 15" (375mm) diameter where access openings exceed 50' (15m). The minimum size of pipe permitted within the VDOT right-of-way is 15" (375mm) unless it is the initial pipe in the system or as a lateral line when necessary. The initial pipe or lateral line in the VDOT right-of-way may be 12” (300mm) provided there is 50 feet or less between access points.

6-0902.2B (42-94-PFM) Pipes shall be designed for flows intercepted by the inlets, with a minimum design for the 10-yr storm.

6-0902.2C Pipes 18" (450mm) in diameter and larger may be constructed on horizontal curves. Tables 6.10 thru 6.16 and Plates 10-6 (10M-6), 16-6 (16M-6) through 18-6 (18M-6) provide for the geometric limitation and information to assist in the design of concrete pipes on horizontal curves.

6-0902.2D (88-05-PFM) Except where noted differently under § 6-0902.2A, the maximum length between access openings shall not exceed 300' (91m) for pipes less than or equal to 42" (1050mm) in diameter or 800' (244m) for pipes greater than 42" (1050mm) in diameter. Access opening may be in the form of an inlet, manhole, junction box or other approved appurtenance.

6-0902.2E Prefabricated "T" and "Y" sections may be used under the conditions stated in the "T" and "Y" Standard (Plate 9-6(9M-6)) when approved by the Director.

6-0902.2F Prefabricated bend sections for storm drainage systems are not permitted within VDOT rights-of-way. Any change in horizontal and vertical alignment shall require an access opening. An exception to this is the horizontal curve shown on Plate 16-6 (16M-6). However, use of prefabricated reinforced concrete "T" and "Y" sections shown in Plate 9-6 (9M-6) and bends shown on Plate 10-6 (10M-6) may be approved by the Director.

6-0902.2G (47-95-PFM) In general, there may not be a reduction in pipe size greater than one standard increment along the direction of flow. Within VDOT maintained rights-of-way, reductions may only be allowed when determined by VDOT to be appropriate.

6-0902.2H Minimum cover for storm sewer pipe shall be 24" (600mm) from finish grade to the outside top of pipe, except where approved structural correction is provided when cover requirements cannot be met.

6-0902.2I Minimum easement widths shall be determined as follows:

TABLE 6.8 MINIMUM EASEMENT WIDTHS/PIPE SIZE

Pipe Size, in. (mm)	Easement Width, ft (m)
15 – 18 (375 – 450)	10 (3.0)
21 – 33 (525 – 825)	15 (4.6)
36 – 48 (900 – 1200)	20 (6.1)
54 – 72 (1350 – 1800)	24 (7.3)

Where multiple pipes are installed, the edge of the easement shall be 5' (1.5m) clear of outside of pipe. Where easements do not generally follow established lot lines, add 5' (1.5m) to the easement width on the side toward the building. No storm drain pipe shall be installed within 5' (1.5m) of the loading plane of a building foundation. Storm sewers to be maintained by DPWES shall be within dedicated storm drainage easements.

6-0902.2J Storm sewers shall be designed to provide an average velocity when running full of not less than 2 ½ FPS (0.76 MPS).

6-0902.2K The need for concrete anchors must be investigated on storm sewer lines with grades of 20% or greater. If anchors are required, the design engineer shall show a detail on the plans with spacing requirements.

6-0902.2L Plain concrete pipe shall conform to the requirements of ASTM Designation C-14 Extra Strength; reinforced concrete pipe shall conform to ASTM Designation C-76 Classes II, III, and IV; a minimum of Class III or equal is required under areas subject to vehicular traffic. Asbestos cement pipe shall conform to ASTM C663-73a, Type II and AASHTO M-217-75.

6-0902.2M When storm sewers are provided, they shall not outfall in the front yard of a lot, but shall be extended at least to within 20' (6m) of the rear property line in lots up to ½ acre (2025 m^2) in size and at least 50' (15m) to the rear of the house on larger size lots. If the storm sewer outfalls on a lot or adjacent to a lot, on which a building exists which will remain, the building must be shown with topography of the area between the building and the outfall. Floor elevations shall be provided , if possible.

6-0902.2N In general, drainage facilities may not be terminated short of the subdivision boundary unless an adequate outfall exists at this point. Deposits may be required for future extension(s) to the subdivision boundary.

6.0902.2O High Density Polyethylene Pipe (HDPE) (78-03-PFM) (101-08-PFM)

6-0902.2O(1) HDPE pipe shall conform to the requirements of AASHTO M 294. The maximum size permitted is 48" (1200 mm).

6-0902.2O(2) Joints shall be watertight meeting a pressure test of 10.8 psi per ASTM D 3212 and use a bell and spigot design with a rubber gasket meeting the requirements of ASTM F 477, "Standard Specification for Elastomeric Seals (Gaskets) for Joining Plastic Pipe." These joints are designed to prevent infiltration of soil and exfiltration of storm water.

6-0902.2O(3) Installations and pipe cover shall be in accordance with ASTM D 2321-"Standard Practice for Underground Installation of Thermoplastic Pipe for Sewers and Other Gravity-Flow Applications", the manufacturer's recommendations and VDOT standards, whichever are more stringent. Pipe bedding and backfill shall conform to the standards set forth in Plate #93-6 (#93M-6).

6-0902.2O(4) Filter fabric shall surround the aggregate fill material when there is a high water table or where the movement of groundwater can cause the migration of fines from the soil envelope. Provide an overlap of 2' (600 mm) minimum. Use non-woven geotextile fabric with AOS of 70-100 US Sieve or 0.22 mm – 0.15 mm as determined by ASTM D 4751 and a trapezoidal tear strength of 45 LB (0.2 kN) as determined by ASTM D 4533. Geotextile fabric shall not be exposed to direct sunlight for more than 24 hours prior to installation.

6-0902.2O(5) The installer shall use flexible waterstops, resilient connections, or other flexible systems to make watertight connections to manholes and other structures in accordance with ASTM F 2510/F 2510M, “Standard Specification for Resilient Connectors Between Reinforced Concrete Manhole Structures and Corrugated High Density Polyethylene Drainage Pipes”, or ASTM C923 “Standard Specifications for Resilient Connectors Between Reinforced Concrete Manhole Structures, Pipes and Laterals” such as A-LOK, KOR-N-Seal, or approved equal. Grouting between the thermoplastic pipe and the manhole and other structures shall not be permitted.

6-0902.2O(6) All pipes shall undergo inspection and deflection testing during and after installation to ensure proper performance in accordance with § 2-0502.

6-0903 Pipe and Culvert Materials. Pipe and culvert materials acceptable for storm drain construction with the accompanying roughness coefficients are shown below:

TABLE 6.9 PIPE AND CULVERT MATERIALS – ROUGHNESS COEFFICIENTS
(78-03-PFM) (101-08-PFM)

Material	Manning "n"
Plain Concrete Culvert Pipe (PCCP)[2]	.013
Non-Reinforced Concrete Sewer Pipe (NRCSP)[2]	.013
Reinforced Concrete Culvert Pipe (RCCP)	.013
Reinforced Concrete Sewer Pipe (RCSP)	.013
Vitrified Clay Pipe, Extra Strength (VSPX)	.013
Cast Iron Pipe (CIP)	.013
Asbestos Cement Pipe (ACP)[3]	.012
Corrugated Plain Metal Pipe (CMP)[1]	.024
25% Paved	.021
50% Paved	.018
100% Paved	.013
High-Density Polyethylene Pipe (HDPE)[4]	.012

[1]Corrugated metal pipe is approved for use at residential driveway entrances, temporary installations, and privately maintained detention systems. Except for the above uses, this type of pipe may be used only when approved by the Director. In approving the use of CMP, the Director may apply certain conditions to provide for inspection and testing in accordance with AASHTO's standards, including deflection testing.

[2]Plain Concrete Culvert Pipe (PCCP) and Non-Reinforced Concrete Sewer Pipe (NRCSP) shall conform to the VDOT Road and Bridge Specifications. Pipe sizes 12" (300mm) through 24" (600mm) are permitted.

[3]VDOT accepts use of asbestos cement pipe under the following conditions:

Maximum size permitted in VDOT's right-of-way is 24" (600mm).

Use is approved by the County.

VDOT shall be furnished details of the location of each installation. An extra set of those sheets of the plan showing the location of the pipe shall be required.

[4]High Density Polyethylene pipe shall conform to the classification Type S.

6-0904 Energy and Hydraulic Gradients. (See Plate 12-6 (12M-6)). The hydraulic gradient for a storm sewer system is a line connecting points to which water will rise in manholes and inlets throughout the system during the design flow. The energy gradient is a line drawn a distance $V^2/2g$ above the hydraulic gradient of the pipes.

6-0904.1 At storm sewer junctions the total energy loss at the junction, H_L, is the difference in elevation between the energy grade lines of the upstream and downstream pipes. To establish these gradients for a system, it is necessary to start at a point where the hydraulic and energy gradients are known or can readily be determined.

6-0904.2 Generally, when the energy and hydraulic gradients must be determined, the pipes are assumed to have uniform flow. For uniform gravity flow and for pressure flow, the friction loss in storm sewer pipes may be determined by the Manning Formula as follows:

$h_f = SL = [(nV)^2/2.208r^{1.33}]\ L$
$(h_f = SL = [(nV)^2/r^{1.33}]\ L)$

h_f = Friction loss in pipe in ft (m)
S = Slope of the energy grade line
n = Roughness coefficient
V = Discharge velocity in FPS (MPS)
r = Hydraulic radius in ft (m)
L = Length of line in ft (m)

6-0904.3 Few design situations will ever require determination of energy and hydraulic gradients for non-uniform flow conditions. Should non-uniform flow analysis be necessary, designers are referred to standard hydraulic texts for determining gradients for non-uniform flow.

6-0904.4 Where a proposed drainage system is connected to an existing drainage system the hydraulic gradient at the point of junction shall be determined from the hydraulic gradient computation of the existing system on file with DPWES.

6-0904.5 The total energy losses at a junction, H_L, is assumed to be made up of 1 or more of the following losses:

6-0904.5A Expansion loss, h_i, when stormwater enters the junction.

6-0904.5B Contraction loss, h_o, when stormwater leaves the junction.

6-0904.5C Bend loss, $h_\in$, due to the change in horizontal direction of stormwater velocity.

These losses may be estimated as follows:

$$H_L = h_i + h_o + h_\Delta = 0.1\frac{V_i^2}{2g} + .05\frac{V_o^2}{2g} + K_\Delta \frac{V_i 2}{2g}$$

H_L = Total Energy Loss
h_i = Expansion Loss (flow in to junction)
h_o = Contraction Loss (flow out of junction)
$h_\in$ = Bend Loss
V_i = Velocity in FPS (MPS), Q/A, of upstream pipe
V_o = Velocity in FPS (MPS), Q/A, of downstream pipe
$\in$ = Horizontal angle in degrees between the direction of flow of incoming and outgoing pipes
$K_\in$ = Bend loss coefficient (see Plates 13-6 (13M-6) and 14-6 (14M-6)

6-0904.6 Considerable judgement must be used when applying the above energy loss equations. Some general rules to be used when applying these equations are as follows:

6-0904.6A When 2 or more pipes discharge into a manhole or inlet type structure, the expansion loss for the junction shall be calculated for the pipe discharge that produces the maximum momentum.

6-0904.6B When 2 or more pipes discharge into a manhole or inlet type structure at different angles of flow with the outgoing pipe, the junction bend loss shall be calculated for the pipe discharge that produces the maximum momentum.

6-0904.6C Prefabricated "T", "Y", and bend sections are assumed to have bend losses only.

Momentum may be determined as follows:

M = Q(w/g)V

M = Momentum
Q = Pipe discharge CFS (CMS)
w/g = Density of water 62.4 lbs/ft^3 (1000 kg/m^3)
V = Discharge velocity in FPS (MPS)

6-0904.7 Since the density of water can be considered constant, the pipe discharge with the largest product, QV, will have the maximum momentum.

6-0904.8 The energy loss for the initial inlet(s) of a storm sewer system may be assumed to be 0.3 times the velocity head in the outlet pipe.

6-0904.9 The above energy loss formulas can be readily solved with the use of Plate 14-6 (14M-6) and a transparency made to conform to Plate 13-6 (13M-6).

6-0904.10 Non-pressure Flow. Storm sewer systems generally shall be designed as non-pressure systems. In general, if a drop in the structure between the inverts of the incoming and outgoing pipes is approximated by a value equal to or greater than the junction energy loss, the system can be assumed to be non-pressure flow.

6-0904.11 Pressure Flow. Storm sewer systems may be designed for pressure flow; however, all proposed pressure flow systems should be coordinated with DPWES in the preliminary design stage. The hydraulic gradient for the design flows shall be at least 1' (0.3m) below the established ground elevation and no more than 5' (1.5m) above the crown of the pipe. For curb opening inlets the gutter flow line is considered the established ground elevation.

6-0904.12 Drop. If possible the energy losses through a junction should be accounted for by a drop across the junction. The equations on Plate 15-6 (15M-6) show the method for computing the drop.

6-0905 Closed Conduit Design Calculations. In general, design calculations required for submittal to the Director are as follows:

6-0905.1 A copy of the drainage plan showing drainage divides, contributing areas and adopted Comprehensive Plan recommendation or existing zoning, whichever is higher.

6-0905.2 Stormwater runoff quantities.

6-0905.3 Pipe design calculations:

6-0905.3A For storm sewer systems or portions of systems designed for pressure flow, a storm sewer profile with energy and hydraulic gradients drawn on it shall be submitted for the portion of the system that experiences pressure flow.

6-0905.3B Energy and hydraulic gradients do not need to be submitted for non-pressure systems.

6-0905.4 Energy loss calculations at storm sewer junctions.

6-0906 Minimum Radius of Curvature for Concrete Pipeline

The following 7 tables are based on Plate 16-6 (16M-6) equations (1) and (2) and present the radius of curvature for joint openings from 1/8" (3mm) through 1 1/2" (38mm). As illustrated in Plate 16-6 (16M-6), Figure 2, when concrete pipe is installed on curved alignment using deflected straight pipe, the point of curve (PC) is at the midpoint of the last undeflected pipe section and the point of tangent (PT) is at the midpoint of the last pulled pipe.

Table 6.10 Radius of Curvature for Straight Deflected Pipe Length of 4' (1.2m)

JOINT OPENING* IN INCHES

	in	1/8	1/4	3/8	1/2	5/8	3/4	7/8	1	1-1/8	1-1/4	1-3/8	1-1/2	in	
	18	736	368	245	184	147	123	105	92	82	74	67	61	18	
	21	848	424	283	212	170	141	121	106	94	85	77	71	21	
	24	960	480	320	240	192	160	137	120	107	96	87	80	24	
	27	1072	536	357	268	214	179	153	134	119	107	97	89	27	
	30	1184	592	395	296	237	197	169	148	132	118	107	99	30	
	33	1296	648	432	324	259	216	185	162	144	130	118	108	33	
P	36	1408	704	469	352	282	235	201	176	156	141	128	117	36	P
I	42	1632	816	544	408	326	272	233	204	181	163	148	136	42	I
P	48	1856	928	619	464	371	309	265	232	206	186	169	155	48	P
E	54	2080	1040	693	520	416	347	297	260	231	208	189	173	54	E
	60	2304	1152	768	576	461	384	329	288	256	230	209	192	60	
D	66	2528	1264	843	632	506	421	361	316	281	253	230	211	66	D
I	72	2752	1376	917	688	550	459	393	344	306	275	250	229	72	I
A	78	2976	1488	992	744	595	496	425	372	331	298	271	248	78	A
M	84	3200	1600	1067	800	640	533	457	400	356	320	291	267	84	M
E	90	3434	1712	1141	856	685	571	489	428	380	342	311	285	90	E
T	96	3648	1824	1216	912	730	608	521	456	405	365	332	304	96	T
E	102	3872	1936	1291	968	774	645	553	484	430	387	352	323	102	E
R	108	4096	2048	1365	1024	819	683	585	512	455	410	372	341	108	R
	114	4256	2128	1419	1064	851	709	608	532	473	426	387	355	114	
	120	4480	2240	1493	1120	896	747	640	560	498	448	407	373	120	
	126	4704	2352	1568	1176	941	784	672	588	523	470	428	392	126	
	132	4928	2464	1643	1232	986	821	704	616	548	493	448	411	132	
	138	5152	2576	1717	1288	1030	859	736	644	572	515	468	429	138	
	144	5376	2688	1792	1344	1075	896	768	672	597	538	489	448	144	

JOINT OPENING* IN MILLIMETERS

	mm	3	6	10	13	16	19	22	25	29	32	35	38	mm	
	450	224.3	112.2	74.8	56.1	44.9	37.4	32.0	28.0	24.9	22.4	20.4	18.7	450	
	525	258.5	129.2	86.2	64.6	51.7	43.1	36.9	32.3	28.7	25.8	23.5	21.5	525	
	600	292.6	146.3	97.5	73.1	58.5	48.8	41.8	36.6	32.5	29.3	26.6	24.4	600	
	675	326.7	163.4	108.9	81.7	65.3	54.5	46.7	40.8	36.3	32.7	29.7	27.2	675	
	750	360.9	180.4	120.3	90.2	72.2	60.1	51.6	45.1	40.1	36.1	32.8	30.1	750	
	825	395.0	197.5	131.7	98.8	79.0	65.8	56.4	49.4	43.9	39.5	35.9	32.9	825	
P	900	429.2	214.6	143.1	107.3	85.8	71.5	61.3	53.6	47.7	42.9	39.0	35.8	900	P
I	1050	497.4	248.7	165.8	124.4	99.5	82.9	71.1	62.2	55.3	49.7	45.2	41.4	1050	I
P	1200	565.7	282.9	188.6	141.4	113.1	94.3	80.8	70.7	62.9	56.6	51.4	47.1	1200	P
E	1350	634.0	317.0	211.3	158.5	126.8	105.7	90.6	79.2	70.4	63.4	57.6	52.8	1350	E
	1500	702.3	351.1	234.1	175.6	140.5	117.0	100.3	87.8	78.0	70.2	63.8	58.5	1500	
D	1650	770.5	385.3	256.6	192.6	154.1	128.4	110.1	96.3	85.6	77.1	70.0	64.2	1650	D
I	1800	838.8	419.4	279.6	209.7	167.8	139.8	119.8	104.8	93.2	83.9	76.3	69.9	1800	I
A	1950	907.1	453.5	302.4	226.8	181.4	151.2	129.6	113.4	100.8	90.7	82.5	75.6	1950	A
M	2100	975.4	487.7	325.1	243.8	195.1	162.6	139.3	121.9	108.4	97.5	88.7	81.3	2100	M
E	2250	1043.6	521.8	347.9	260.9	208.7	173.9	149.1	130.5	116.0	104.4	94.9	87.0	2250	E
T	2400	1111.9	556.0	370.6	278.0	222.4	185.3	158.8	139.0	123.5	111.2	101.1	92.7	2400	T
E	2550	1180.2	590.1	393.4	295.0	236.0	196.7	168.6	147.5	131.1	118.0	107.3	98.3	2550	E
R	2700	1248.5	624.2	416.2	312.1	249.7	208.1	178.4	156.1	138.7	124.8	113.5	104.0	2700	R
	2850	1297.2	648.6	432.4	324.3	259.4	216.2	185.3	162.2	144.1	129.7	117.9	108.1	2850	
	3000	1365.5	682.8	455.2	341.4	273.1	227.6	195.1	170.7	151.7	136.5	124.1	113.8	3000	
	3150	1433.8	716.9	477.9	358.4	286.8	239.0	204.8	179.2	159.3	143.4	130.3	119.5	3150	
	3300	1502.1	751.0	500.7	375.5	300.4	250.3	214.6	187.8	166.9	150.2	136.5	125.2	3300	
	3450	1570.3	785.2	523.4	392.6	314.1	261.7	224.3	196.3	174.5	157.0	142.8	130.9	3450	
	3600	1638.6	819.3	546.2	409.7	327.7	273.1	234.1	204.8	182.1	163.9	149.0	136.5	3600	

***Joint opening is the pulled distance from the home (or normal) position and the deflected position. Reference Standard No. RC-1**

Table 6.11 Radius of Curvature for Straight Deflected Pipe Length of 6' (1.8m)

JOINT OPENING* IN INCHES

	in	1/8	1/4	3/8	1/2	5/8	3/4	7/8	1	1-1/8	1-1/4	1-3/8	1-1/2	in	
	18	1104	552	368	276	221	184	158	138	123	110	100	92	18	
	21	1272	636	424	318	254	212	182	159	141	127	116	106	21	
	24	1440	720	480	360	288	240	206	180	160	144	131	120	24	
	27	1608	804	536	402	322	268	230	201	179	161	146	134	27	
	30	1776	888	592	444	355	296	254	222	197	178	161	148	30	
	33	1944	972	648	486	389	324	278	243	216	194	177	162	33	
P	36	2112	1056	704	528	422	352	302	264	235	211	192	176	36	P
I	42	2448	1224	816	612	490	408	350	306	272	245	223	204	42	I
P	48	2784	1392	928	696	557	464	398	348	309	278	253	232	48	P
E	54	3120	1560	1040	780	624	520	446	390	347	312	284	260	54	E
	60	3456	1728	1152	864	691	576	494	432	384	346	314	288	60	
D	66	3792	1896	1264	946	758	632	542	474	421	379	345	316	66	D
I	72	4128	2064	1376	1032	826	688	590	516	459	413	375	344	72	I
A	78	4464	2232	1488	1116	893	744	638	558	496	446	406	372	78	A
M	84	4800	2400	1600	1200	960	800	686	600	533	480	436	400	84	M
E	90	5136	2568	1712	1284	1027	856	734	642	571	514	467	428	90	E
T	96	5472	2736	1824	1368	1094	912	782	684	608	547	497	456	96	T
E	102	5808	2904	1936	1452	1162	968	830	726	645	581	528	484	102	E
R	108	6144	3072	2048	1536	1229	1024	878	768	683	614	559	512	108	R
	114	6384	3192	2128	1596	1277	1064	912	798	709	638	580	532	114	
	120	6720	3360	2240	1680	1344	1120	960	840	747	672	611	560	120	
	126	7056	3528	2352	1764	1411	1176	1008	882	784	706	641	588	126	
	132	7392	3696	2464	1848	1478	1232	1056	924	821	739	672	616	132	
	138	7728	3864	2576	1932	1546	1288	1104	966	859	773	703	644	138	
	144	8064	4032	2688	2016	1613	1344	1152	1008	896	806	733	672	144	

JOINT OPENING* IN MILLIMETERS

	mm	3	6	10	13	16	19	22	25	29	32	35	38	mm	
	450	336.5	168.2	112.2	84.1	67.3	56.1	48.1	42.1	37.4	33.6	30.6	28.0	450	
	525	387.7	193.9	129.2	96.9	77.5	64.6	55.4	48.5	43.1	38.8	35.2	32.3	525	
	600	438.9	219.5	146.3	109.7	87.8	73.1	62.7	54.9	48.8	43.9	39.9	36.6	600	
	675	490.1	245.1	163.4	122.5	98.0	81.7	70.0	61.3	54.4	49.0	44.5	40.8	675	
	750	541.3	270.7	180.4	135.3	108.3	90.2	77.3	67.7	60.1	54.1	49.2	45.1	750	
	825	592.5	296.3	197.5	148.1	118.5	98.8	84.6	74.1	65.8	59.2	53.9	49.4	825	
P	900	643.7	321.9	214.6	160.9	128.7	107.3	92.0	80.5	71.5	64.4	58.5	53.6	900	P
I	1050	746.1	373.1	248.7	186.5	149.2	124.4	106.6	93.3	82.9	74.6	67.8	62.2	1050	I
P	1200	848.6	424.3	282.9	212.1	169.7	141.4	121.2	106.1	94.3	84.9	77.1	70.7	1200	P
E	1350	951.0	475.5	317.0	237.7	190.2	158.5	135.9	118.9	105.7	95.1	86.4	79.2	1350	E
	1500	1053.4	526.7	351.1	263.3	210.7	175.6	150.5	131.7	117.0	105.3	95.8	87.8	1500	
D	1650	1155.8	577.9	385.3	288.9	231.2	192.6	165.1	144.5	128.4	115.6	105.1	96.3	1650	D
I	1800	1258.2	629.1	419.4	314.6	251.6	209.7	179.7	157.3	139.8	125.8	114.4	104.8	1800	I
A	1950	1360.6	680.3	453.5	340.2	272.1	226.8	194.4	170.1	151.2	136.1	123.7	113.4	1950	A
M	2100	1463.0	731.5	487.7	365.8	292.6	243.8	209.9	182.9	162.6	146.3	133.0	121.9	2100	M
E	2250	1565.5	782.7	521.8	391.4	313.1	260.9	223.6	195.7	173.9	156.5	142.3	130.5	2250	E
T	2400	1667.9	833.9	556.0	417.0	333.6	278.0	238.3	208.5	185.3	166.8	151.6	139.0	2400	T
E	2550	1770.3	885.1	590.1	442.6	354.1	295.0	252.9	221.3	196.7	177.0	160.9	147.5	2550	E
R	2700	1872.7	936.3	624.2	468.2	374.5	312.1	267.5	234.1	208.1	187.3	170.2	156.1	2700	R
	2850	1945.8	972.9	648.6	486.5	389.2	324.3	278.0	243.2	216.2	194.6	176.9	162.2	2850	
	3000	2048.3	1024.1	682.8	512.1	409.7	341.4	292.6	256.0	227.6	204.8	186.2	170.7	3000	
	3150	2150.7	1075.3	716.9	537.7	430.1	358.4	307.2	268.8	239.0	215.1	195.5	179.2	3150	
	3300	2253.1	1126.5	751.0	563.3	450.6	375.5	321.9	281.6	250.3	225.3	204.8	187.8	3300	
	3450	2355.5	1177.7	785.2	588.9	471.1	392.6	336.5	294.4	261.7	235.5	214.1	196.3	3450	
	3600	2457.9	1229.0	819.3	614.5	491.6	409.7	351.1	307.2	273.1	245.8	223.4	204.8	3600	

***Joint opening is the pulled distance from the home (or normal) position and the deflected position. Reference Standard No. RC-1**

Table 6.12 Radius of Curvature for Straight Deflected Pipe Length of 7-1/2' (2.3m)

JOINT OPENING* IN INCHES

	in	1/8	1/4	3/8	1/2	5/8	3/4	7/8	1	1-1/8	1-1/4	1-3/8	1-1/2	in	
	18	1380	690	460	345	276	230	197	172	153	138	125	115	18	
	21	1590	795	530	397	318	265	227	199	177	159	145	132	21	
	24	1800	900	600	450	360	300	257	225	200	180	164	150	24	
	27	2010	1005	670	502	402	335	287	251	223	201	183	167	27	
	30	2220	1110	740	555	444	370	317	277	247	222	202	185	30	
	33	2430	1215	810	607	486	405	347	304	270	243	221	202	33	
P	36	2640	1320	880	660	528	440	377	330	293	264	240	220	36	P
I	42	3060	1530	1020	765	612	510	437	382	340	306	278	255	42	I
P	48	3480	1740	1160	870	696	580	497	435	387	348	316	290	48	P
E	54	3900	1950	1300	975	780	650	557	487	433	390	355	325	54	E
	60	4320	2160	1440	1080	864	720	617	540	480	432	393	360	60	
D	66	4740	2370	1580	1185	948	790	677	592	527	474	431	395	66	D
I	72	5160	2580	1720	1290	1032	860	737	645	573	516	469	430	72	I
A	78	5580	2790	1860	1395	1116	930	797	697	620	558	507	465	78	A
M	84	6000	3000	2000	1500	1200	1000	857	750	667	600	545	500	84	M
E	90	6420	3210	2140	1605	1284	1070	917	802	713	642	584	535	90	E
T	96	6840	3420	2280	1710	1368	1140	977	855	760	684	622	570	96	T
E	102	7260	3630	2420	1815	1452	1210	1037	907	807	726	660	605	102	E
R	108	7680	3840	2560	1920	1536	1280	1097	960	853	768	698	640	108	R
	114	7980	3990	2660	1995	1596	1330	1140	997	887	798	725	665	114	
	120	8400	4200	2800	2100	1680	1400	1200	1050	933	840	764	700	120	
	126	8820	4410	2940	2205	1764	1470	1260	1102	980	882	802	735	126	
	132	9240	4620	3080	2310	1848	1540	1320	1155	1027	924	840	770	132	
	138	9660	4830	3220	2415	1932	1610	1380	1207	1073	966	878	805	138	
	144	10080	5040	3360	2520	2016	1680	1440	1260	1120	1008	916	840	144	

JOINT OPENING* IN MILLIMETERS

	mm	3	6	10	13	16	19	22	25	29	32	35	38	mm	
	450	420.6	210.3	140.2	105.1	84.1	70.1	60.1	52.6	46.7	42.0	38.2	35.0	450	
	525	484.6	242.3	161.5	121.2	96.9	80.8	69.2	60.6	53.8	48.4	44.0	40.4	525	
	600	548.6	274.3	182.9	137.2	109.7	91.4	78.4	68.6	60.9	54.9	49.9	45.7	600	
	675	612.6	306.3	204.2	153.2	122.5	102.1	87.5	76.6	68.1	61.3	55.7	51.0	675	
	750	676.7	338.3	225.5	169.2	135.3	112.8	96.7	84.6	75.2	67.7	61.5	56.4	750	
	825	740.7	370.3	246.9	185.2	148.1	123.4	105.8	92.6	82.3	74.1	67.3	61.7	825	
P	900	804.7	402.3	268.2	201.2	160.9	134.1	114.9	100.6	89.4	80.5	73.1	67.0	900	P
I	1050	932.7	466.3	310.9	233.2	186.5	155.4	133.2	116.6	103.6	93.3	84.8	77.7	1050	I
P	1200	1060.7	530.4	353.6	265.2	212.1	176.8	151.5	132.6	117.9	106.1	96.4	88.4	1200	P
E	1350	1188.7	594.4	396.2	297.2	237.7	198.1	169.8	148.6	132.1	118.9	108.1	99.1	1350	E
	1500	1316.7	658.4	438.9	329.2	263.3	219.5	188.1	164.6	146.3	131.7	119.7	109.7	1500	
D	1650	1444.8	722.4	481.6	361.2	288.9	240.8	206.4	180.6	160.5	144.5	131.3	120.4	1650	D
I	1800	1572.8	786.4	524.3	393.2	314.6	262.1	224.7	196.6	174.7	157.3	143.0	131.1	1800	I
A	1950	1700.8	850.4	566.9	425.2	340.2	283.5	243.0	212.6	189.0	170.1	154.6	141.7	1950	A
M	2100	1828.8	914.4	609.6	457.2	365.8	304.8	261.3	228.6	203.2	182.9	166.3	152.4	2100	M
E	2250	1956.8	978.4	652.3	489.2	391.4	326.1	279.5	244.6	217.4	195.7	177.9	163.1	2250	E
T	2400	2084.8	1042.4	694.9	521.2	417.0	347.5	297.8	260.6	231.6	208.5	189.5	173.7	2400	T
E	2550	2212.8	1106.4	737.6	553.2	442.6	368.8	316.1	276.6	245.9	221.3	201.2	184.4	2550	E
R	2700	2340.9	1170.4	780.3	585.2	468.2	390.1	334.4	292.6	260.1	234.1	212.8	195.1	2700	R
	2850	2432.3	1216.2	810.8	608.1	486.5	405.4	347.5	304.0	270.3	243.2	221.1	202.7	2850	
	3000	2560.3	1280.2	853.4	640.1	512.1	426.7	365.8	320.0	284.5	256.0	232.8	213.4	3000	
	3150	2688.3	1344.2	896.1	672.1	537.7	448.1	384.0	336.0	298.7	268.8	244.4	224.0	3150	
	3300	2816.4	1408.2	938.8	704.1	563.3	469.4	402.3	352.0	312.9	281.6	256.0	234.7	3300	
	3450	2944.4	1472.2	981.5	736.1	588.9	490.7	420.6	368.0	327.2	294.4	267.7	245.4	3450	
	3600	3072.4	1536.2	1024.1	768.1	614.5	512.1	438.9	384.0	341.4	307.2	279.3	256.0	3600	

***Joint opening is the pulled distance from the home (or normal) position and the deflected position. Reference Standard No. RC-1**

Table 6.13 Radius of Curvature for Straight Deflected Pipe Length of 8' (2.4m)

JOINT OPENING* IN INCHES

	in	1/8	1/4	3/8	1/2	5/8	3/4	7/8	1	1-1/8	1-1/4	1-3/8	1-1/2	in	
	18	1472	736	491	368	294	245	210	184	164	147	134	123	18	
	21	1696	848	565	424	339	283	242	212	188	170	154	141	21	
	24	1920	960	640	480	384	320	274	240	213	192	175	160	24	
	27	2144	1072	715	536	429	357	306	268	238	214	195	189	27	
	30	2368	1184	789	592	474	395	338	296	263	237	215	197	30	
	33	2592	1296	864	648	518	432	370	324	288	259	236	216	33	
P	36	2816	1408	939	704	563	469	402	352	313	282	256	235	36	P
I	42	3264	1632	1088	816	653	544	466	408	363	326	297	272	42	I
P	48	3712	1856	1237	928	742	619	530	464	412	371	337	310	48	P
E	54	4160	2080	1387	1040	832	693	594	520	462	416	378	347	54	E
	60	4608	2304	1536	1152	922	768	658	576	512	461	419	384	60	
D	66	5056	2528	1685	1264	1011	843	722	632	562	506	460	421	66	D
I	72	5504	2752	1835	1376	1101	917	786	688	612	550	500	459	72	I
A	78	5952	2976	1984	1488	1190	992	850	744	661	595	541	496	78	A
M	84	6400	3200	2133	1600	1280	1067	914	780	711	640	582	533	84	M
E	90	6848	3424	2283	1712	1370	1141	978	856	761	685	623	571	90	E
T	96	7296	3648	2432	1824	1459	1216	1042	912	811	730	663	608	96	T
E	102	7744	3872	2581	1936	1549	1291	1106	968	860	774	704	645	102	E
R	108	8192	4096	2731	2048	1638	1365	1170	1024	910	819	745	683	108	R
	114	8512	4256	2837	2128	1702	1419	1216	1064	946	851	774	709	114	
	120	8960	4480	2987	2240	1792	1493	1230	1120	996	896	815	747	120	
	126	9480	4704	3136	2352	1882	1568	1344	1176	1045	941	855	784	126	
	132	9856	4928	3285	2464	1971	1643	1408	1232	1095	986	896	821	132	
	138	10300	5152	3435	2576	2061	1717	1472	1288	1145	1030	937	859	138	
	144	10750	5376	3584	2688	2151	1792	1536	1344	1195	1075	977	896	144	

JOINT OPENING* IN MILLIMETERS

	mm	3	6	10	13	16	19	22	25	29	32	35	38	mm	
	450	448.7	224.3	149.6	112.2	89.7	74.8	64.1	56.1	49.8	44.8	40.8	37.4	450	
	525	516.9	258.5	172.3	129.2	103.4	86.1	73.8	64.6	57.4	51.7	47.0	43.1	525	
	600	585.2	292.6	195.1	146.3	117.0	97.5	83.6	73.1	65.0	58.5	53.2	48.8	600	
	675	653.5	326.7	217.8	163.4	130.7	108.9	93.3	81.7	72.6	65.3	59.4	54.4	675	
	750	721.8	360.9	240.6	180.4	144.3	120.3	103.1	90.2	80.2	72.2	65.6	60.1	750	
	825	790.0	395.0	263.3	197.5	158.0	131.7	112.9	98.7	87.8	79.0	71.8	65.8	825	
P	900	858.3	429.2	286.1	214.6	171.7	143.0	122.6	107.3	95.4	85.8	78.0	71.5	900	P
I	1050	994.9	497.4	331.6	248.7	199.0	165.8	142.1	124.4	110.5	99.5	90.4	82.9	1050	I
P	1200	1131.4	565.7	377.1	282.9	226.3	188.6	161.6	141.4	125.7	113.1	102.8	94.3	1200	P
E	1350	1268.0	634.0	422.7	317.0	253.6	211.3	181.1	158.5	140.9	126.8	115.3	105.7	1350	E
	1500	1404.5	702.3	468.2	351.1	280.9	234.1	200.6	175.6	156.1	140.4	127.7	117.0	1500	
D	1650	1541.1	770.5	513.7	385.3	308.2	256.8	220.1	192.6	171.2	154.1	140.1	128.4	1650	D
I	1800	1677.6	838.8	559.2	419.4	335.5	279.6	239.7	209.7	186.4	167.8	152.5	139.8	1800	I
A	1950	1814.2	907.1	604.7	453.5	362.8	302.4	259.2	226.8	201.6	181.4	164.9	151.2	1950	A
M	2100	1950.7	975.4	650.2	487.7	390.1	325.1	278.7	243.8	216.7	195.1	177.3	162.6	2100	M
E	2250	2087.3	1043.6	695.8	521.8	417.5	347.9	298.2	260.9	231.9	208.7	189.7	173.9	2250	E
T	2400	2223.8	1111.9	741.3	556.0	444.8	370.6	317.7	278.0	247.1	222.4	202.2	185.3	2400	T
E	2550	2360.4	1180.2	786.8	590.1	472.1	393.4	337.2	295.0	262.3	236.0	214.6	196.7	2550	E
R	2700	2496.9	1248.5	832.3	642.2	499.4	416.4	356.7	312.1	277.4	249.7	227.0	208.1	2700	R
	2850	2594.5	1297.2	864.8	648.6	518.9	432.4	370.6	324.3	288.3	259.4	235.9	216.2	2850	
	3000	2731.0	1365.5	910.3	682.8	546.2	455.2	390.1	341.4	303.4	273.1	248.3	227.6	3000	
	3150	2867.6	1433.8	955.9	716.9	573.5	477.9	409.6	358.4	318.6	286.8	260.7	239.0	3150	
	3300	3004.1	1502.1	1001.4	751.0	600.8	500.7	429.2	375.5	333.8	300.4	273.1	250.3	3300	
	3450	3140.7	1570.3	1046.9	785.2	628.1	523.4	448.7	392.6	349.0	314.1	285.5	261.7	3450	
	3600	3277.2	1638.6	1092.4	819.3	655.4	546.2	468.2	409.6	364.1	327.7	297.9	273.1	3600	

***Joint opening is the pulled distance from the home (or normal) position and the deflected position. Reference Standard No. RC-1**

Table 6.14 Radius of Curvature for Straight Deflected Pipe Length of 10' (3m)

JOINT OPENING* IN INCHES

	in	1/8	1/4	3/8	1/2	5/8	3/4	7/8	1	1-1/8	1-1/4	1-3/8	1-1/2	in	
	18	1840	920	613	460	368	307	263	230	204	184	167	153	18	
	21	2120	1060	707	530	424	353	303	265	236	212	193	177	21	
	24	2400	1200	800	600	480	400	343	300	267	240	218	200	24	
	27	2680	1340	893	670	536	447	383	335	298	268	244	223	27	
	30	2960	1480	987	740	592	493	423	370	329	296	269	247	30	
	33	3240	1620	1080	810	648	540	463	405	360	324	295	270	33	
P	36	3520	1760	1173	880	704	587	503	440	391	352	320	293	36	P
I	42	4080	2040	1360	1020	816	680	583	510	453	408	371	340	42	I
P	48	4640	2320	1547	1160	928	773	663	580	516	464	422	387	48	P
E	54	5200	2600	1733	1300	1040	867	743	650	578	520	473	433	54	E
	60	5760	2880	1920	1440	1152	960	823	720	640	576	524	480	60	
D	66	6320	3160	2107	1580	1264	1053	908	790	702	632	575	527	66	D
I	72	6880	3440	2293	1720	1376	1147	983	860	764	688	625	573	72	I
A	78	7440	3720	2480	1860	1488	1240	1063	930	827	744	676	620	78	A
M	84	8000	4000	2667	2000	1600	1333	1143	1000	889	800	727	667	84	M
E	90	8560	4280	2853	2140	1712	1427	1223	1070	951	856	778	713	90	E
T	96	9120	4550	3040	2280	1824	1520	1303	1140	1013	912	829	760	96	T
E	102	9680	4840	3227	2420	1936	1613	1383	1210	1076	968	880	807	102	E
R	108	10240	5120	3413	2560	2048	1707	1463	1280	1138	1024	931	853	108	R
	114	10640	5320	3547	2660	2128	1773	1520	1330	1182	1064	967	887	114	
	120	11200	5600	3733	2800	2240	1867	1600	1400	1244	1120	1018	933	120	
	126	11760	5880	3920	2940	2352	1960	1680	1470	1307	1176	1069	980	126	
	132	12320	6160	4107	3080	2464	2053	1760	1540	1369	1232	1120	1027	132	
	138	12880	6440	4293	3220	2576	2147	1840	1610	1431	1288	1171	1073	138	
	144	13440	6720	4480	3360	2688	2240	1920	1680	1493	1344	1222	1120	144	

JOINT OPENING* IN MILLIMETERS

	mm	3	6	10	13	16	19	22	25	29	32	35	38	mm	
	450	560.8	280.4	186.9	140.2	112.2	93.5	80.1	70.1	62.3	56.1	51.0	46.7	450	
	525	646.2	323.1	215.4	161.5	129.2	107.7	92.3	80.8	71.8	64.6	58.7	53.8	525	
	600	731.5	365.8	243.8	182.9	146.3	121.9	104.5	91.4	81.3	73.1	66.5	60.9	600	
	675	816.9	408.4	272.3	204.2	163.4	136.1	116.7	102.1	90.7	81.7	74.2	68.1	675	
	750	902.2	451.1	300.7	225.5	180.4	150.4	128.9	112.8	100.2	90.2	82.0	75.2	750	
	825	987.5	493.8	329.2	246.9	197.5	164.6	141.1	123.4	109.7	98.7	89.8	82.3	825	
P	900	1072.9	536.4	357.6	268.2	214.6	178.8	153.3	134.1	119.2	107.3	97.5	89.4	900	P
I	1050	1243.6	621.8	414.5	310.9	248.7	207.3	177.6	155.4	138.2	124.3	113.0	103.6	1050	I
P	1200	1414.3	707.1	471.4	353.6	282.9	235.7	202.0	176.8	157.1	141.4	128.6	117.8	1200	P
E	1350	1585.0	792.5	528.3	396.2	317.0	264.2	226.4	198.1	176.1	158.5	144.1	132.1	1350	E
	1500	1755.6	877.8	585.2	438.9	351.1	292.6	250.8	219.5	195.1	175.6	159.6	146.3	1500	
D	1650	1926.3	963.2	642.1	481.6	385.3	321.1	275.2	240.8	214.0	192.6	175.1	160.5	1650	D
I	1800	2097.0	1048.5	699.0	524.3	419.4	349.5	299.6	262.1	233.0	209.7	190.6	174.7	1800	I
A	1950	2267.7	1133.9	755.9	566.9	453.5	377.9	324.0	283.5	252.0	226.8	206.2	189.0	1950	A
M	2100	2438.4	1219.2	812.8	609.6	487.7	406.4	348.3	304.8	270.9	243.8	221.7	203.2	2100	M
E	2250	2609.1	1304.5	869.7	652.3	521.8	434.8	372.7	326.1	289.9	260.9	237.2	217.4	2250	E
T	2400	2779.8	1389.9	926.6	694.9	556.0	463.3	397.1	347.5	308.9	278.0	252.7	231.6	2400	T
E	2550	2950.5	1475.2	983.5	737.6	590.1	491.7	421.5	368.8	327.8	295.0	268.2	245.9	2550	E
R	2700	3121.2	1560.6	1040.4	780.3	624.2	520.2	445.9	390.1	346.8	312.1	283.7	260.1	2700	R
	2850	3243.1	1621.5	1081.0	810.8	648.6	540.5	463.3	405.4	360.3	324.3	294.8	270.3	2850	
	3000	3413.8	1706.9	1137.9	853.4	682.8	569.0	487.7	426.7	379.3	341.4	310.3	284.5	3000	
	3150	3584.4	1792.2	1194.8	896.1	716.9	597.4	512.1	448.1	398.3	358.4	325.9	298.7	3150	
	3300	3755.1	1877.6	1251.7	938.8	751.0	625.9	536.4	469.4	417.2	375.5	341.4	312.9	3300	
	3450	3925.8	1962.9	1308.6	981.5	785.2	654.3	560.8	490.7	436.2	392.6	356.9	327.1	3450	
	3600	4096.5	2048.3	1365.5	1024.1	819.3	682.8	585.2	512.1	455.2	409.6	372.4	341.4	3600	

***Joint opening is the pulled distance from the home (or normal) position and the deflected position. Reference Standard No. RC-1**

Table 6.15 Radius of Curvature for Straight Deflected Pipe Length of 12' (3.7m)

JOINT OPENING* IN INCHES

	in	1/8	1/4	3/8	1/2	5/8	3/4	7/8	1	1-1/8	1-1/4	1-3/8	1-1/2	in	
	18	2208	1104	736	552	442	368	315	276	245	221	201	184	18	
	21	2544	1272	848	636	509	424	363	318	283	254	231	212	21	
	24	2880	1440	960	720	576	480	411	360	320	288	262	240	24	
	27	3216	1608	1072	804	643	536	459	402	357	322	292	268	27	
	30	3552	1778	1184	888	710	592	507	444	395	355	323	296	30	
	33	3888	1944	1296	972	778	648	555	486	432	389	353	324	33	
P	36	4224	2112	1408	1056	845	704	603	528	469	422	384	352	36	P
I	42	4896	2448	1632	1224	979	816	699	612	544	490	445	408	42	I
P	48	5568	2784	1856	1392	1114	928	795	696	619	557	506	464	48	P
E	54	6240	3120	2080	1560	1248	1040	891	780	693	624	567	520	54	E
	60	6912	3456	2304	1728	1382	1152	987	864	768	691	628	576	60	
D	66	7584	3792	2528	1896	1517	1264	1083	948	843	758	689	632	66	D
I	72	8256	4128	2752	2064	1651	1376	1179	1032	917	826	751	688	72	I
A	78	8928	4464	2976	2232	1786	1488	1275	1116	992	893	812	744	78	A
M	84	9600	4800	3200	2400	1920	1600	1371	1200	1067	960	873	800	84	M
E	90	10270	5136	3424	2568	2054	1712	1467	1284	1141	1027	934	856	90	E
T	96	10940	5472	3648	2736	2189	1824	1563	1368	1216	1094	995	912	96	T
E	102	11620	5808	3872	2904	2323	1936	1659	1452	1291	1162	1056	968	102	E
R	108	12290	6144	4096	3072	2458	2048	1755	1536	1365	1229	1117	1024	108	R
	114	12770	6384	4256	3192	2554	2128	1824	1596	1419	1277	1161	1064	114	
	120	13440	6720	4480	3360	2688	2240	1920	1680	1493	1344	1222	1120	120	
	126	14110	7056	4704	3528	2822	2352	2016	1764	1568	1411	1283	1176	126	
	132	14780	7392	4928	3696	2957	2464	2112	1848	1643	1478	1344	1232	132	
	138	15460	7728	5152	3864	3091	2576	2208	1932	1717	1548	1405	1288	138	
	144	16130	8064	5376	4032	3226	2688	2304	2016	1792	1613	1466	1344	144	

JOINT OPENING* IN MILLIMETERS

	mm	3	6	10	13	16	19	22	25	29	32	35	38	mm	
	450	673.0	336.5	224.3	168.2	134.6	112.2	96.1	84.1	74.8	67.3	61.2	56.1	450	
	525	775.4	387.7	258.5	193.8	155.1	129.2	110.8	96.9	86.1	77.5	70.5	64.6	525	
	600	877.8	438.9	292.6	219.4	175.6	146.3	125.4	109.7	97.5	87.8	79.8	73.1	600	
	675	980.2	490.1	326.7	245.1	196.0	163.4	140.0	122.5	108.9	98.0	89.1	81.7	675	
	750	1082.6	541.3	360.9	270.7	216.5	180.4	154.7	135.3	120.3	108.2	98.4	90.2	750	
	825	1185.1	592.5	395.0	296.3	237.0	197.5	169.3	148.1	131.7	118.5	107.7	98.7	825	
P	900	1287.5	643.7	429.2	321.9	257.5	214.6	183.9	160.9	143.0	128.7	117.0	107.3	900	P
I	1050	1492.3	746.1	497.4	373.1	298.5	248.7	213.2	186.5	165.8	149.2	135.7	124.3	1050	I
P	1200	1697.1	848.6	565.7	424.3	339.4	282.8	242.4	212.1	188.6	169.7	154.3	141.4	1200	P
E	1350	1902.0	951.0	634.0	475.5	380.4	317.0	271.7	237.7	211.3	190.2	172.9	158.5	1350	E
	1500	2106.8	1053.4	702.3	526.7	421.4	351.1	301.0	263.3	234.1	210.7	191.5	175.6	1500	
D	1650	2311.6	1155.8	770.5	577.9	462.3	385.3	330.2	288.9	256.8	231.2	210.1	192.6	1650	D
I	1800	2516.4	1258.2	838.8	629.1	503.3	419.4	359.5	314.5	279.6	251.6	228.8	209.7	1800	I
A	1950	2721.3	1360.6	907.1	680.3	544.2	453.5	388.7	340.2	302.4	272.1	247.4	226.8	1950	A
M	2100	2926.1	1463.0	975.4	731.5	585.2	487.7	418.0	365.8	325.1	292.6	266.0	243.8	2100	M
E	2250	3130.9	1565.6	1043.6	782.7	626.2	521.8	447.3	391.4	347.9	313.1	284.6	260.9	2250	E
T	2400	3335.7	1667.9	1111.9	833.9	667.1	556.0	476.5	417.0	370.6	333.6	303.2	278.0	2400	T
E	2550	3540.6	1770.3	1180.2	885.1	708.1	590.1	505.8	442.6	393.4	354.1	321.9	295.0	2550	E
R	2700	3745.4	1872.7	1248.5	936.3	749.1	624.2	535.1	468.2	416.1	374.5	340.5	312.1	2700	R
	2850	3891.7	1945.8	1297.2	972.9	778.3	648.6	556.0	486.5	432.4	389.2	353.8	324.3	2850	
	3000	4096.5	2048.3	1365.5	1024.1	819.3	682.7	585.2	512.1	455.2	409.6	372.4	341.4	3000	
	3150	4301.3	2150.7	1433.8	1075.3	860.3	716.9	614.5	537.6	477.9	430.1	391.0	358.4	3150	
	3300	4506.2	2253.1	1502.1	1126.5	901.2	751.0	643.7	563.3	500.7	450.6	409.6	375.5	3300	
	3450	4711.0	2355.5	1570.3	1177.7	942.2	785.2	673.0	588.9	523.4	471.1	428.3	392.6	3450	
	3600	4915.8	2457.9	1638.6	1229.0	983.2	819.3	702.3	614.5	546.2	491.6	446.9	409.6	3600	

***Joint opening is the pulled distance from the home (or normal) position and the deflected position. Reference Standard No. RC-1**

Table 6.16 Radius of Curvature for Straight Deflected Pipe Length of 16' (4.9m)

JOINT OPENING* IN INCHES

	in	1/8	1/4	3/8	1/2	5/8	3/4	7/8	1	1-1/8	1-1/4	1-3/8	1-1/2	in	
	18	2944	1472	981	736	539	491	421	368	327	294	268	245	18	
	21	3392	1696	1131	848	678	565	485	424	377	339	308	283	21	
	24	3840	1920	1280	960	768	640	549	480	427	384	349	320	24	
	27	4288	2144	1429	1072	858	715	613	536	476	429	390	357	27	
	30	4736	2368	1579	1184	947	789	677	592	526	474	430	395	30	
	33	5184	2592	1728	1296	1037	864	741	648	576	518	471	432	33	
P	36	5632	2816	1877	1408	1126	939	805	704	626	563	512	469	36	P
I	42	6528	3254	2176	1632	1306	1088	933	816	725	653	593	544	42	I
P	48	7424	3712	2475	1856	1485	1237	1061	928	825	743	675	619	48	P
E	54	8320	4160	2773	2080	1664	1387	1189	1040	924	832	756	693	54	E
	60	9216	4628	3072	2304	1843	1536	1317	1152	1024	922	838	768	60	
D	66	10110	5056	3371	2528	2022	1685	1445	1264	1124	1011	919	843	66	D
I	72	11010	5504	3669	2752	2202	1835	1573	1376	1223	1101	1001	917	72	I
A	78	11900	5952	3968	2976	2381	1984	1701	1488	1323	1190	1082	992	78	A
M	84	12800	6400	4267	3200	2560	2133	1829	1600	1422	1280	1164	1067	84	M
E	90	13700	6848	4565	3424	2739	2283	1957	1712	1522	1370	1245	1141	90	E
T	96	14590	7296	4864	3648	2918	2432	2085	1824	1621	1459	1327	1216	96	T
E	102	15490	7744	5163	3872	3098	2581	2213	1936	1721	1549	1408	1291	102	E
R	108	16380	8192	5461	4096	3277	2731	2341	2048	1820	1638	1489	1365	108	R
	114	17020	8512	5675	4256	3405	2837	2432	2128	1892	1702	1548	1419	114	
	120	17920	8960	5973	4480	3584	2987	2560	2240	1991	1792	1629	1493	120	
	126	18820	9408	6272	4704	3763	3136	2688	2352	2091	1882	1711	1568	126	
	132	19710	9856	6671	4928	3942	3285	2816	2464	2190	1971	1792	1643	132	
	138	20610	10300	6869	5152	4122	3435	2944	2576	2290	2061	1873	1717	138	
	144	21500	10750	7168	5376	4301	3584	3072	2688	2389	2150	1955	1792	144	

JOINT OPENING* IN MILLIMETERS

	mm	3	6	10	13	16	19	22	25	29	32	35	38	mm	
	450	897.3	448.7	299.1	224.3	179.4	149.5	128.2	112.1	99.7	89.7	81.5	74.7	450	
	525	1033.9	516.9	344.6	258.5	206.8	172.3	147.7	129.2	114.8	103.4	94.0	86.1	525	
	600	1170.4	585.2	390.1	292.6	234.1	195.1	167.2	146.3	130.0	117.0	106.4	97.5	600	
	675	1307.0	653.5	435.7	326.7	261.4	217.8	186.7	163.4	145.2	130.7	118.8	108.9	675	
	750	1443.5	721.8	481.2	360.9	288.7	240.6	206.2	180.4	160.4	144.3	131.2	120.3	750	
	825	1580.1	790.0	526.7	395.0	316.0	263.3	225.7	197.5	175.5	158.0	143.6	131.7	825	
P	900	1716.6	858.3	572.2	429.2	343.3	286.1	245.2	214.6	190.7	171.6	156.0	143.0	900	P
I	1050	1989.7	994.9	663.2	497.4	397.9	331.6	284.2	248.7	221.1	199.0	180.9	165.8	1050	I
P	1200	2262.8	1131.4	754.3	565.7	452.6	377.1	323.3	282.8	251.4	226.3	205.7	188.6	1200	P
E	1350	2535.9	1268.0	845.3	634.0	507.2	422.6	362.3	317.0	281.8	253.6	230.5	211.3	1350	E
	1500	2809.0	1404.5	936.3	702.3	561.8	468.2	401.3	351.1	312.1	280.9	255.4	234.1	1500	
D	1650	3082.1	1541.1	1027.4	770.5	616.4	513.7	440.3	385.3	342.5	308.2	280.2	256.8	1650	D
I	1800	3355.2	1677.6	1118.4	838.8	671.0	559.2	479.3	419.4	372.8	335.5	305.0	279.6	1800	I
A	1950	3628.3	1814.2	1209.4	907.1	725.7	604.7	518.3	453.5	403.1	362.8	329.8	302.4	1950	A
M	2100	3901.4	1950.7	1300.5	975.4	780.3	650.2	557.3	487.7	433.5	390.1	354.7	325.1	2100	M
E	2250	4174.5	2087.3	1391.5	1043.6	834.9	695.8	596.4	521.8	463.8	417.4	379.5	347.9	2250	E
T	2400	4447.6	2223.8	1482.5	1111.9	889.5	741.3	635.4	555.9	494.2	444.8	404.3	370.6	2400	T
E	2550	4720.7	2360.4	1573.6	1180.2	944.1	786.8	674.4	590.1	524.5	472.1	429.2	393.4	2550	E
R	2700	4993.8	2496.9	1664.6	1248.5	998.8	832.3	713.4	624.2	554.9	499.4	454.0	416.1	2700	R
	2850	5188.9	2594.5	1729.6	1297.2	1037.8	864.8	741.3	648.6	576.5	518.9	471.7	432.4	2850	
	3000	5462.0	2731.0	1820.7	1365.5	1092.4	910.3	780.3	682.7	606.9	546.2	496.5	455.2	3000	
	3150	5735.1	2867.6	1911.7	1433.8	1147.0	955.8	819.3	716.9	637.2	573.5	521.4	477.9	3150	
	3300	6008.2	3004.1	2002.7	1502.1	1201.6	1001.4	858.3	751.0	667.6	600.8	546.2	500.7	3300	
	3450	6281.3	3140.7	2093.8	1570.3	1256.3	1046.9	897.3	785.2	697.9	628.1	571.0	523.4	3450	
	3600	6554.4	3277.2	2184.8	1638.6	1310.9	1092.4	936.3	819.3	728.3	655.4	595.9	546.2	3600	

***Joint opening is the pulled distance from the home (or normal) position and the deflected position. Reference Standard No. RC-1.**

6-1000 OPEN CHANNELS

6-1001 Water Surface Profiles (Standard Step Method and Direct Step Method)

6-1001.1 Water surface profiles for steady flow in non-uniform channels with frequent changes of cross-section and grade, and uniform channels with frequent changes of grade cannot be computed accurately by assuming uniform flow conditions where there is no appreciable length of constant section and grade and there is no opportunity for conditions of uniform flow to exist.

6-1001.2 Water surface profiles for steady flow in channels of this type are determined by computing separately and successively the change in surface elevation in each of a number of small portions of the total length of the profile.

6-1001.2A These small portions, called reaches, must be short enough to reduce to a permissible magnitude the error in approximating the true slope of the water surface profile through the reach by the average of the surface slopes at each end, or by the slope corresponding to the average of the hydraulic properties of the reach.

6-1001.2B These reaches shall be selected with due regard to the irregularities in the channel.

6-1001.2C The step computations are carried upstream if the flow is subcritical and downstream if the flow is supercritical.

6-1001.3 Various textbooks and publications on open-channel hydraulics explain in detail the step method of computing water surface profiles for prismatic and non-prismatic channels.

6-1001.4 Flow profiles by The Standard Step Method and Direct-Step Method have been programmed for the electronic computer. Employment of these programs should not be attempted without a knowledge of energy balance and without prior experience of working flow profiles by the manual method.

6-1002 Side Ditches and Median Ditches

6-1002.1 As the necessity arises, special procedures and nomographs are developed to be used to facilitate repetitive design. Charts of the side and median ditch series have been developed to facilitate and simplify the design (See VDOT Drainage Manual).

6-1002.2 Side and Median Ditch Design. Follow the general procedure outlined below (see also § 6-1010):

6-1002.2A Note on the computation form, under Station to Station, points at 100' (30m) intervals where roadside ditches, median ditches or valleys, formed by fill slope and inward sloping existing ground, will be constructed.

6-1002.2B Note, by flow arrow on the form, the direction the flow will take in the side ditch.

6-1002.2C Note the average width of the strip to be drained. Use of the cross-sections, contour maps or aerial photos will facilitate this operation.

6-1002.2D Determine the design discharge, for each 100' (30m) interval point, starting at the first point down grade from the peak in the ditch grade and proceeding down grade. The following method of determining this "Q" will suffice: Compile a table of CA values that will cover the various width strips.

6-1002.2D(1) Multiply the appropriate CA value, or the sum of the appropriate CA values, by the rainfall intensity. The rainfall intensity will decrease approximately 0.1" (2.5mm) for each additional 100' (30m) the flow travels in the ditch.

6-1002.2D(2) The resultant "Q" is entered in the space provided on the form.

6-1002.2E Note the slope of the ditch flow line in the space provided on the form.

6-1002.2F Enter the appropriate Side Ditch Flow Chart with "Q" and slope to determine the velocity of flow using "n"=0.030 for unpaved ditches.

6-1002.2G Where the velocity, as determined above, exceeds the allowable velocity, as determined from the soil classification in the soils report, the ditch shall be lined.

6-1002.2G(1) To determine the depth of flow in the lined ditch, enter the appropriate Side Ditch Flow

Chart, using the appropriate "n" with "Q" and slope and read the depth of flow.

6-1002.2G(2) Standard paved ditches or special design paved ditches, gutters or channels having "D" dimension sufficient to cover the majority of the maximum depths noted on the computation form shall be required where the computations indicate the maximum allowable velocity is exceeded.

6-1002.2G(3) The "D" dimension shall be noted on the plans along with the standard used.

6-1002.2G(4) A typical section of all special design paved ditches, gutters, or channels shall be included in the plans.

6-1002.2G(5) Paved ditch construction specifications are shown in § 6-1013 and Plate 19-6 (19M-6) shows standard ditch sections. The ditch sections transitioning from full width to yard inlet are shown on Plate 20-6 (20M-6).

6-1002.2H A typical example will be found in § 6-1009, 6-1010 and 6-1011.

6-1003 Channel Charts. The trapezoidal channel charts of the CC series have been developed to supplement the Federal Highway Administration Publication (see VDOT Drainage Manual).

6-1004 Design Criteria

6-1004.1 In general, roadside and median ditches shall be designed with sufficient capacity to contain the runoff for a 10-yr storm. For determining whether or not special linings will be required and the lining dimensions, the 2-yr storm shall be used.

6-1004.1A (61-98-PFM) For an engineered grass swale, ditch or channel designed to convey stormwater within County easements provided for swales, ditches or channels, the maximum design velocity (V) shall be no greater than 4FPS (1.2 MPS), as determined by the formula cited in § 6-1005.1. Swales, ditches or channels exceeding these parameters will require special linings. This requirement does not apply to emergency spillways for dams. Vegetated spillway velocity requirements are included in § 6-1600.

6-1004.1B (61-98-PFM) All special channels shall be designed for storm frequencies in accordance with the importance of the road and its vulnerability to inundation, should the capacity be exceeded.

6-1004.1C (61-98-PFM) If the newly constructed channel (ditch) alongside, or leading from, any street providing access to lots to be occupied, or through, or alongside any such lots, is not well stabilized within 120 days after initial attempts to stabilize, or 120 days after issuance of any Residential or Non-Residential Use Permit for such lots, whichever occurs first, the channel (ditch) must be paved.

6-1004.2 In the event an exception for a winter Residential Use Permit is granted as provided for in Paragraph 2 of § 18-704 of the Zoning Ordinance, the 120 days shall run from March 15 of the following spring. "Well stabilized" shall mean a good stand of grass must be growing and not showing any visible evidence of erosive forces. Sod shall be growing well and knitted into the underlying soil.

6-1005 Channel Size and Shape

6-1005.1 The size of a channel shall primarily be established by the Manning Formula which may be expressed as:

$$Q = VA = 1.49/n \ r^{2/3} \ S^{1/2} \ A$$
$$(Q = VA = 1/n \times r^{2/3} \times S^{1/2} \times A)$$

Definitions of the terms are given in § 6-0902 (see Plate 27-6 (27M-6) and Table 6.17).

6-1005.2 General guidelines related to the size and shape of channels are:

6-1005.2A Low flow sections should be considered in the design of channels with large cross-sections.

6-1005.2B Channel bottom of widths greater than 10' (3m) shall be built with a minimum cross slope of 1:12.

6-1005.2C The side slopes of a channel shall be a function of channel material. The side slopes throughout the entire length of a channel shall be stable.

6-1005.2D Channels to be constructed on horizontal curves should be investigated to see if channel section needs to be modified. The superelevation of the water in the channel may be computed by:

e = V^2 w/gr

e = Difference in elevation between the water surface on the inside and outside walls of the channel in ft (m).

V = Mean velocity of flow in FPS (MPS).

w = Average width of channel in ft (m).

g = Acceleration of gravity, 32.2 FPS^2 (9.81 MPS^2)

r = Radius of channel centerline in ft (m). The rise in water surface may be accounted for by channel freeboard and/or superelevation of the channel sides.

6-1005.2E Minimum easement widths shall be determined as follows:

TABLE 6.16A
MINIMUM EASEMENT WIDTHS – CHANNELS

Top Width of Channel	Easement Width
< 2' (< 0.6m)	10' (3m)
2' – 4' (0.6m – 1.2m)	10' (3m) greater than top width of channel with minimum of 5' (1.5m) on 1 side.
> 4' (> 1.2m)	15' (4.6m) greater than top width of channel with minimum of 5' (1.5m) on 1 side.

Channels to be maintained by DPWES shall be within dedicated storm drainage easements.

6-1006 Channel Materials. Channel materials acceptable for open channel design with the accompanying roughness coefficients are shown below:

TABLE 6.17
CHANNEL MATERIALS – "n"

Material	n
Concrete, trowel finish	0.013
Concrete, broom or float finish	0.015
Gunite	0.018
Riprap placed (VDOT Class I)	0.030
Riprap dumped (VDOT Class I)	0.035
Gabion	0.028

6-1007 Energy and Hydraulic Gradients. (Reference Plates 24-6 (24M-6) through 26-6 (26M-6))

6-1007.1 The hydraulic gradient for an open channel system is the water surface. The energy gradient is a line drawn a distance $V^2/2g$ above the hydraulic gradient. At channel junctions, the total energy loss at the junction, H_L, is the difference in elevation between the energy grade lines of the upstream and downstream channels. To establish these gradients for a system, it is necessary to start at a point where the energy and hydraulic gradients are known or can readily be determined.

6-1007.2 Generally, when the energy and hydraulic gradients must be determined, the channels are assumed to have uniform flow. For uniform flow the friction loss along the channel may be determined by the Manning Formula as discussed above and in § 6-0902.

6-1007.2A Energy Loss at Channel Transitions. The energy loss for open channel transitions may be calculated by:

h_l = $k_l(V^2/2g)$

h_l = Energy loss at transitions due to change in flow area, slope, roughness or any combination of the characteristics.

$V^2/2g$ = Change in velocity head before and after the transitions. This value is always considered positive.

k_1 = 0.2 for channel expansion, i.e., velocities decreasing along direction of flow.

k_1 = 0.1 for channel contraction, i.e., velocities increasing along direction of flow.

Some general guidelines to the design of channel transitions are as follows:

6-1007.2A(1) Transition to channel connections should be connected with smooth tangent type surfaces.

6-1007.2A(2) A straight line connecting flow lines at the 2 ends of the transition should not make an angle greater than 12 1/2° with the axis of the channel.

6-1007.2A(3) Make transition length considerably greater than transition width.

6-1007.2B Energy Loss through Horizontal Channel Curve.

In addition to the friction loss through a channel curve, there is an additional energy loss due to the change in direction of flow. This loss may be calculated as follows:

h_2 = $k_2(V^2/2g)$

h_2 = Energy loss in a curved channel due to change in direction of flow.

k_2 = Energy loss coefficient which may be determined from Plate 25-6 (25M-6).

$V^2/2g$ = Velocity head in curve.

6-1007.2C Drop. If possible the energy losses through a transition or horizontal curve should be accounted for by an increase in channel slope through the transition and/or curve. The equations above and Plate 26-6 (26M-6) show the method for computing the drop.

6-1008 Channel Design Calculations. In general the following design calculations shall be required for submission of plans to the County:

6-1008.1 Design flows shall be determined by methods discussed in § 6-0800 et seq.

6-1008.2 Plans showing channels carrying flows no greater than 30 CFS (0.85 CMS) shall show channel capacity calculations.

6-1008.3 Plans showing channels carrying flows 30 CFS (0.85 CMS) and greater shall show:

6-1008.3A Channel capacity calculations.

6-1008.3B Calculations showing that freeboard requirements have been met.

6-1008.3C Energy and hydraulic gradients drawn on storm sewer profiles at channel transitions and/or curves.

6-1008.4 A note stating that "All grass-lined channels must be in a well stabilized condition and show no signs of erosion at the time of final acceptance by the maintaining authority" shall be shown on all applicable plans.

6-1009 Example – Paved Ditch Computations. Given or assumed (values vary with projects):

6-1009.1 Q=CIA; where C=0.9 for paved area, C=0.5 for unpaved drainage area within normal rights-of-way, C=0.3 for drainage area outside normal rights-of-way. "I" is based on the 2-yr rainfall curve with time of concentration dependent upon average width, grade and type of cover, (5% and average grass in this case).

$$A = \frac{100 \times \text{Width Strip}}{43{,}560}$$

A = area in acres
Width Strip = width in ft.

6-1009.2 Typical Section: 24' pavement, ditch having 3:1 front slope and 2:1 back slope.

6-1009.3 (91-06-PFM) From Virginia Erosion and Sediment Control Handbook, Chapter 5, mostly silt loam with a short section of ordinary firm load.

6-1009.4 (91-06-PFM) Allowable Velocity: From Table 5-22 in the Virginia Erosion and Sediment Control Handbook use 3 FPS as permissible velocity for silt loam and 3.4 FPS for ordinary firm loam.

6-1009.5 Normal right-of-way width = 110'.

6-1009.6 Width Strip Drained: To be determined from cross-sections, aerial photographs, topographical sheets or field observation (to be measured from outside edge of pavement to the nearest multiple of 10').

6-1009.7 (61-98-PFM) Where vegetative linings are used, n=0.050 should be used and a velocity of 4 FPS should be the upper permitted maximum.

6-1010 Example - Paved Ditch Computations. "C" "A" Values for 100' of ditch, using various widths and roughness factors.

		Col. 1 No Pavement	Col. 1 + 0.025* 12' Pavement	Col. 1 + 0.050** 24' Pavement
$\frac{30 \times 100 \times 0.5}{43,560}$	=	0.035	0.060	0.085
$\frac{40 \times 100 \times 0.5}{43,560}$	=	0.046	0.071	0.096
$\frac{60 \times 100 \times 0.48}{43,560}$	=	0.066	0.091	0.116
$\frac{100 \times 100 \times 0.41}{43,560}$	=	0.094	0.119	0.144
$\frac{150 \times 100 \times 0.37}{43,560}$	=	0.128	0.153	0.178
$\frac{200 \times 100 \times 0.35}{43,560}$	=	0.161	0.186	0.211

***12' Pavement Computations**

$$\frac{12 \times 100 \times 0.9}{43,560} = 0.025$$

****24' Pavement Computations**

$$\frac{24 \times 100 \times 0.9}{43,560} = 0.050$$

Note: See § 6-1002 and VDOT Drainage Manual Section 2.7.3.

From 2-yr Curve – RAINFALL

Duration (minutes)	6	7	8	9	10	11	12	13	14	15
Intensity	4.8	4.6	4.4	4.3	4.1	4.0	3.9	3.7	3.6	3.5

TABLE 6.18 TIME OF CONCENTRATION TO USE – PAVED DITCH

30'	Width Strip	-	t_c	6 minutes,	I 4.8 in./hr
40'	Width Strip	-	t_c	7 minutes,	I 4.6 in./hr
60'	Width Strip	-	t_c	9 minutes,	I 4.3 in./hr
100'	Width Strip	-	t_c	10 minutes,	I 4.1 in./hr
150'	Width Strip	-	t_c	12 minutes,	I 3.9 in./hr
200'	Width Strip	-	t_c	14 minutes,	I 3.6 in./hr

6-1011 Example – Paved Ditch Computations. Decrease "I" value 0.1 in./hr for each additional 100' that water flows in the ditch.

Time of Concentration is based on Plate 4-6.

COMPUTATIONS

Sta. 136 + 00 to 142 + 00 and Sta. 149 + 50 to 157 + 50

Check Point	C A Values	CAI = Q
Sta. 137+00	0.060	0.060 x 4.8 = 0.2880 CFS
Sta. 138+00	0.071 0.131	0.131 x 4.6 = 0.6026 CFS
Sta. 139+00	0.119 0.250	0.250 x 4.1 = 1.0250 CFS
Sta. 140+00	0.119 0.369	0.369 x 4.0 = 1.4760 CFS
Sta. 141+00	0.071 0.440	0.440 x 3.9 = 1.7160 CFS
Sta. 142+00	0.071 0.511	0.511 x 3.8 = 1.9418 CFS
Sta. 156+50	0.096	0.096 x 4.6 = 0.6228 CFS
Sta. 155+50	0.116 0.212	0.212 x 4.3 = 0.9116 CFS
Sta. 154+50	0.144 0.356	0.356 x 4.1 = 1.4596 CFS
Sta. 153+50	0.211 0.567	0.567 x 3.6 = 2.0412 CFS
Sta. 152+50	0.211 0.778	0.778 x 3.5 = 2.7230 CFS
Sta. 151+50	0.178 0.956	0.956 x 3.4 = 3.2504 CFS
Sta. 150+50	0.119 1.075	1.075 x 3.3 = 3.5475 CFS
Sta. 149+50	0.091 1.166	1.166 x 3.2 = 3.7312 CFS

6-1012 Paved Ditch Construction Specifications (renumbered by 91-06-PFM)

6-1012.1 All construction and materials shall conform, where applicable, to the current VDOT Road and Bridge Specifications except as noted herein:

6-1012.1A The Director may require special designs for paved ditches as he deems necessary.

6-1012.1B The dimensions shown on the typical section are minimum.

6-1012.1C The concrete shall be A3 (Class 20).

6-1012.1D The subgrade shall be constructed to the required elevation below the finished surface of the paved ditch in accordance with the dimensions and design as shown on the approved plans.

6-1012.1E All soft and unsuitable materials shall be removed and replaced with an approved material which shall be compacted to 95% density in accordance with AASHTO-99-61 and finished to a smooth surface.

6-1012.1F The subgrade shall be moistened prior to the placing of the concrete.

6-1012.1G Ditches shall be formed to true typical section in accordance with the alignment dimensions and design required by the approved plans.

6-1012.1H All forms shall be inspected before the placing of concrete.

6-1012.1I A minimum 6" (150mm) diameter underdrain shall be placed where excessive ground water conditions are encountered to limits as deemed necessary by the Director.

6-1012.1J Underdrains shall be encased in washed gravel.

6-1012.1K On curves, the paved ditch shall be formed on the specified curve as indicated on the approved plans.

6-1012.1L The finish surface of the paved ditch shall be coarse or roughened texture.

6-1012.1M 4" (100mm) weep holes shall be provided as directed by the inspector.

6-1012.1N A minimum of 1 ft^3 ($0.03m^3$) of 2" (50mm) washed gravel shall be placed at the mouth of each drain pipe.

6-1012.1O The type, dimensions (WxBxD), and limits shall be indicated on the plans.

6-1012.1P In the case of special designs, the plans will indicate a typical section with dimensions and the limits to be provided.

6-1012.1Q All transitions shall be shown on the plans and the limits indicated.

6-1012.1R Ditches shall be reinforced with 6" x 6" No. 6 (152 x 152 - MW 19 x MW 19) welded wire fabric. The welded wire fabric and reinforcing steel, when required, shall conform to the current VDOT Road and Bridge Specifications.

6-1012.1S PD-A, B, C & D ditches shown on Plate 19-6 (19M-6) shall be poured in alternate sections of 10' (3m) and no section shall be less than 5' (1.5m). Construction joints shall be provided every 10' (3m) and ¾" (19mm) bituminous expansion material shall be provided every 40' (12m) and shall extend to full depth of slab. The expansion joint filler shall conform to the current VDOT Road and Bridge Specifications.

6-1012.1T Curtain walls shall be provided at each end of the paved ditch, and at other locations where undermining can occur. This curtain wall shall extend a minimum of 18" (450mm) below and perpendicular to the grade of the paved ditch. It shall be as thick as the concrete thickness of the ditch slab.

6-1012.1U (47-95-PFM) Paved ditches constructed of asphalt concrete shall not be permitted.

6-1012.1V Gabions may be used in lieu of paved ditches when approval has been given by the Director. These gabions will be of the Maccaferri or Bekaert type or approved equivalent. Typical gabion uses for channel section, revetment with toe wall and weir section are shown in Plates 21-6 (21M-6) thru 23-6 (23M-6).

6-1100 STORM SEWER APPURTENANCES

6-1101 General

6-1101.1 Wherever possible storm sewer appurtenances should conform with the standards shown in this § 6-0000 et seq., or the current VDOT Drainage Manual. Special designs are subject to approval by the Director.

6-1101.2 Storm sewer appurtenances shall be designed for the runoff generated by the 10-yr frequency storm as determined by the methods discussed in § 6-0800 et seq. Standard specifications for storm sewers are in § 6-1110.

6-1102 (88-05-PFM) **Curb Inlets in VDOT Right-of-way.** Curb inlets to be maintained by VDOT shall be designed in accordance with the VDOT Drainage Manual. The spread of water on the pavement shall be limited to the width of one-half of the travel lane and the gutter width in each direction or 8 to 10 feet from the face of curb, whichever is less, for a rainfall intensity of 4 inches per hour. Under certain conditions, the drop inlet may need to be designed and/or checked using a rainfall intensity of 6.5 inches per hour (See Chapter 9 of the VDOT Drainage Manual for the applicability of this requirement).

6-1103 (88-05-PFM) **Curb Inlets on Private Streets or Parking Lots.** The length of curb inlet opening is dependent on the inlet location, pavement, geometry, and the amount of flow approaching the inlet. General guidelines pertaining to design of curb inlets in private streets and parking lots are as follows:

6-1103.1 (88-05-PFM) Water shall be picked up on continuous grades of curb and gutter streets with projected traffic volumes of 1000 or less ADT before the spread into the street exceeds 15' (4.5m).

6-1103.2 (88-05-PFM) Water shall be picked up on continuous grades of curb and gutter streets with projected traffic volumes of greater than 1000 ADT before the spread into the street exceeds 12' (3.6m).

6-1103.3 (88-05-PFM) Inlets on continuous grades may be designed with a percentage of the flow bypassing the inlet. Bypass flow must be accounted for at the next downstream inlet.

6-1103.4 (88-05-PFM) The amount of intercept required on continuous grades is governed by the spread of flow into the street.

6-1103.5 (88-05-PFM) Inlets in sumps must be designed to take flow from the area draining toward it and any bypass flow that may occur from upstream inlets.

6-1103.6 (88-05-PFM) Sump inlets located in streets shall be designed so the spread into the street does not exceed 10' (3m) at the low point.

6-1103.6A (88-05-PFM) The spread requirements of 15' (4.5m) or 12' (3.6m) stated in § 6-1103.1 and 6-1103.2 must be met at the point above the sump location where the street grade is 0.2%. The design flow to a sump inlet from each direction must be calculated.

6-1103.6B (88-05-PFM) It is not necessary to adhere to the spread requirements at the 0.2% street grade for inlets at sump locations within the turnaround of a cul-de-sac. However, flow depths and directions and grading must be checked and the turnaround designed to prevent local flooding of adjacent property.

6-1103.6C (88-05-PFM) The amount of flow to the inlet must be checked to see that the flow is not directed at driveway entrances where it could "jump" the curb. Also, overland relief must be checked.

6-1103.6D (88-05-PFM) The minimum length of inlet throat at sump locations shall not be less than 6' (1.8m).

6-1103.7 (88-05-PFM) When street grades are less than 2%, a maximum of 2 CFS (0.057 CMS) may be allowed to cross the intersection of private streets, if the projected traffic volume is equal to or less than 1000 ADT. Flows in excess of 2 CFS (0.057 CMS) but no more than 4 CFS (0.113 CMS) will be allowed to cross intersections of private streets when the grade across the intersection is 2% or greater, if the projected traffic volume is equal to or less than 1000 ADT.

6-1103.8 (88-05-PFM) No flows shall be allowed to cross streets, if the projected traffic volume is greater than 1000 ADT.

6-1103.9 (88-05-PFM) The minimum length of curb inlet throat shall not be less than 2'6" (0.75m).

6-1103.10 (88-05-PFM) Curb inlets in private streets can easily be designed in accordance with the above guidelines by use of the charts in Plates 28-6 (28M-6) and 29-6 (29M-6).

6-1103.11 (88-05-PFM) The capacity of curb inlets and the spread of gutter flow in parking lots will vary considerably if the cross slope to the inlet or curb is significantly different from the standard street cross slope of 1/4":1' (1:50). This variation should be taken into consideration if the cross slope is less than 1/16":1' (1:200) or greater than 1/2":1' (1:25). Plate 30-6 (30M-6) may be used along with Plates 28-6 (28M-6) and 29-6 (29M-6) to account for these differences.

6-1103.12 (88-05-PFM) Plate 31-6 (31M-6) may be used for designing curb inlets located at low points in grade.

6-1103.13 (88-05-PFM) Curb inlets shall not be built within curb returns.

6-1103.14 (88-05-PFM) When plans show the old Fairfax County CI-1 to be built, at the option of the developer, it is requested that it be replaced by a VDOT standard DI-3B, with a 4' (1.2m) throat. Where a CI-2 is shown on the plans, it is requested that it be replaced with a DI-3B with an 8' (2.4m) throat when located on a grade, or with a DI-3C with an 8' (2.4m) throat when located in a sump.

6-1104 Yard Inlets

6-1104.1 (88-05-PFM) The required size of yard inlet openings shall be determined by Plate 31-6 (31M-6).

6-1104.2 (88-05-PFM) Yard inlets should be positioned in such a way that they intercept all the design flow approaching the inlet. This can generally be accomplished by depressing the inlet and/or with use of an earth berm.

6-1104.3 (88-05-PFM) Any area, which is inundated by water ponding at a yard inlet, shall be within the storm drainage easement.

6-1104.4 (88-05-PFM, 52-96-PFM) Yard inlet and typical details are shown on Plates 32-6 (32M-6), 33-6 (33M-6), 72-6 (72M-6), 73-6 (73M-6), 74-6 (74M-6) and 75-6 (75M-6).

6-1105 Frames & Covers

6-1105.1 (88-05-PFM) Frames and covers within VDOT rights-of-way shall conform with VDOT specifications.

6-1105.2 (88-05-PFM) Frames and covers in easements outside VDOT rights-of-way are shown on Plate 34-6 (34M-6).

6-1106 Grate Inlets. (88-05-PFM, 52-96-PFM) The capacity of grate inlets in sumps and on grades may be obtained from Plates 35-6 (35M-6) and 36-6 (36M-6). To allow for clogging, grate inlets used at sump locations shall be sized for 100% more capacity than the design flow (i.e., use 50% clogging factor). Grate inlets may be acceptable in VDOT right-of-way and privately owned and maintained systems. However, grate inlets are not acceptable for use on drainage structures to be maintained by Fairfax County or located in County drainage easements.

6-1107 Open Top Structures. (88-05-PFM) Open top structures such as VDOT's IT-1 shall not be permitted.

6-1108 Energy Dissipation Devices

6-1108.1 (88-05-PFM) The terminal ends of all pipes and paved channel storm sewer systems shall be evaluated to be sure that the receiving surface will experience no erosion due to the design discharge.

6-1108.2 (88-05-PFM) Where design discharges have velocities greater than the erosive velocity of the receiving surface, an energy dissipation device shall be designed or a standard energy dissipation device shall be specified.

6-1108.3 (88-05-PFM, 57-96-PFM) Riprap used for erosion control shall conform to the current version of the VDOT Road and Bridge Specifications.

6-1109 Drainage in Residential Areas

6-1109.1 (88-05-PFM) General guidelines to be observed in drainage design in residential subdivision developments in which the average lot size is less than 18,000 ft² (1672 m²).

6-1109.1A (88-05-PFM) No quantity of design surface runoff across lots shall be erosive.

6-1109.1B (29-89-PFM, 57-96-PFM, 88-05-PFM) Quantities of surface runoff greater than 2 CFS (0.057 CMS) that flow through lots shall be picked up and conveyed in a closed storm drainage system except that the Director may allow the quantity of surface runoff to be increased to 3 CFS (0.085 CMS) where the developer demonstrates that such increase will not result in lot drainage problems. The Director may approve an open channel where the preservation of a natural drainageway is desirable or the use of open channel will not interfere with use of the property. Open channel may also be used where the size of the storm sewer pipe exceeds 72" (1800mm).

6-1109.1C (88-05-PFM) Lots generally shall be graded in such a manner that surface runoff does not cross more than 3 lots before it is collected in a storm sewer system. This system may be open channel, closed conduit, or a combination of both.

6-1109.2 (88-05-PFM) The following general guidelines are to be observed in drainage design in residential subdivision developments in which the average lot size is 18,000 ft² (1672 m²) or greater with ditch section roads:

6-1109.2A (88-05-PFM) No quantity of design surface runoff across lots shall be erosive.

6-1109.2B (88-05-PFM) Drainage from rights-of-way should flow in an easement along lot lines whenever possible.

6-1109.2C (88-05-PFM) Once drainage is concentrated in rights-of-way, it shall be transferred to a logical point of discharge, preferably a storm sewer system, either open channel, closed conduit, or a combination of both.

6-1109.2D (88-05-PFM) In fill sections, a ditch at the toe of a fill may be necessary. If the toe of fill ditch is outside of the right-of-way, it must be in an easement.

6-1109.3 (88-05-PFM) If it cannot be established how drainage concentrated in the rights-of-way ultimately will be handled, the affected lots shall be restricted until such time as a grading plan showing ultimate drainage disposition has been submitted and approved.

6-1109.4 (88-05-PFM) Townhouse Downspout Discharge – Any downspout discharging onto yards (particularly on rear yards) which consist of filled ground, where the runoff will flow over a fill grade of 15% or greater within 30' (9m) of the pipe discharge location, shall have the downspout picked up by an underdrain and carried out to the toe of the fill slope to natural ground.

6-1110 Inlet Design Calculations. (88-05-PFM) In general, design calculations required for submission to the County are as follows:

6-1110.1 (88-05-PFM) Calculations showing that the spread of gutter flow in the street is within the allowable range.

6-1110.2 (88-05-PFM) Calculations showing the percent of interception of gutter flow.

6-1110.3 (88-05-PFM) Capacity calculations for all inlets.

6-1110.4 (88-05-PFM) Evaluation of the terminal ends of piped and paved ditch systems for the possible need of energy dissipation devices.

6-1110.5 (88-05-PFM) The drainage divide sheet shall clearly show both on-site and off-site areas attributing runoff to each inlet.

6-1111 Storm Sewer Construction Specifications

6-1111.1 (88-05-PFM) All construction and materials shall conform where applicable to the current VDOT Road and Bridge Specifications.

6-1111.2 (88-05-PFM) All concrete shall be A3 (Class 20) if cast in place, A4 (Class 30) if precast.

6-1111.3 (88-05-PFM) Drop inlets and curb inlets shall have steps. The maximum dimension from finish grade to the first step in the inlet shall not exceed 3'3" (975mm).

6-1111.4 (88-05-PFM) Unless stated on the approved plans, symmetrical channels shall be formed in the invert of all structures according to VDOT standard IS-1 to prevent standing or ponding of water.

6-1111.5 (88-05-PFM) Manholes and drop inlets shall be constructed from invert to top as follows:

6-1111.5A (88-05-PFM) Manholes to 8' (2.4m) deep

6-1111.5A(1) (88-05-PFM) Block construction – minimum 8" (200mm) walls

6-1111.5A(2) (88-05-PFM) Poured in place concrete – minimum 8" (200mm) walls and non-reinforced

6-1111.5A(3) (88-05-PFM) Pre-cast – minimum 8" (200mm) block walls in conjunction with precast throat and precast base slab.

6-1111.5A(4) (88-05-PFM) Precast

6-1111.5B (88-05-PFM) Manholes over 8' (2.4m) deep

6-1111.5B(1) (88-05-PFM) Precast

6-1111.5B(2) (88-05-PFM) Poured in place reinforced concrete

6-1111.5B(3) (88-05-PFM) Special design, i.e., bends, precast "T", precast boxes

6-1111.6 (88-05-PFM) Inlets where pipe size is larger than 48" (1200mm) ID require a special design. In the case of special design inlets that deviate from the standard, the precast manufacturer or design engineer must submit 5 copies of the detail drawings to the County for approval.

6-1111.7 (88-05-PFM) If block construction is used, the inside and outside walls, as they are laid, shall be plastered with mortar a minimum of ½" (13mm) thick.

6-1111.8 (88-05-PFM) All precast drop inlets, curb inlets and manholes shall conform to the latest edition of ASTM C-478.

6-1111.9 (88-05-PFM) The opening in precast storm sewer structures for all size pipe shall be a minimum of 4" (100mm) and a maximum of 8" (200mm) larger than the outside diameter of the pipe.

6-1111.10 (88-05-PFM) The contractor must notify the design engineer as to which structures will be precast so that the proper stake out procedures can be followed.

6-1111.11 (88-05-PFM) The "H" dimension shown on the standards and specified on the plans will be measured from the invert of the outfall pipe to the top of the structure.

6-1111.12 2" (88-05-PFM) (50mm) weep holes shall be provided in endwalls where directed by the inspector.

6-1111.13 (88-05-PFM) A guardrail, fence or other protective device shall be required when the height of an endwall is 2' (0.6m) or greater and the structure is located near residence or pedestrian walkways. The protective device shall be shown on the plan. Guardrails will be so placed so as to perform the function for which it is intended and the height of the guardrail shall extend 36" (900mm) above the surrounding area.

6-1200 CULVERTS

6-1201 Design Flow

6-1201.1 Culverts shall generally be designed for the 25-yr rainfall frequency when crossing under primary roads, 10-yr rainfall frequency when crossing under secondary roads and other locations.

6-1201.2 Culverts shall be checked for the effects of the 100-yr storm. No flooding of building structures shall result from the 100-yr design flow.

6-1201.3 Design flows shall be determined by methods discussed in § 6-0000 et seq.

6-1202 Size

6-1202.1 Culverts experience 2 major types of flow: Flow with inlet control and flow with outlet control.

6-1202.1A Under outlet control, all of the culvert parameters including the headwater depth, type of inlet, cross-sectional area, slope, roughness, length and tail water elevation influence the culvert size and capacity.

6-1202.1B Under inlet control the capacity of the entire culvert is limited by the capacity of the inlet and only the first 3 of the above parameters are of primary importance.

6-1202.2 The items in § 6-1202.1 shall be taken into consideration when sizing culverts. In general, culverts shall be hydraulically designed in accordance with the US Department of Transportation's latest publication, "Hydraulic Charts for the Selection of Highway Culverts."

6-1202.3 Considerable savings may be realized in designing culverts with improved inlets. These types of culverts may be hydraulically designed in accordance with "Hydraulic Design of Improved Inlets for Culverts," Hydraulics Engineering Circular #13, dated August 1972, which was prepared by the Hydraulic Branch, Bridge Division, Office of the Engineer, in collaboration with the Research Development Demonstration Division, Region 13, Federal Highway Administration, Washington, DC 20590.

6-1202.4 General guidelines in selection of culvert size are as follows:

6-1202.4A Headwater depth for design discharge shall not exceed a height greater than 1 1/2' (0.45m) below the edge of the shoulder of a road.

6-1202.4B In general, maximum allowable headwater above the crown of a culvert shall not be greater than 5' (1.5m).

6-1202.4C Headwater depth for the design discharge shall not cause water to rise above the top of approach channels which are adjacent to improved land or above the established floodplain easements.

6-1202.4D Headwater depth at design discharge shall cause no flooding of existing or proposed building structures.

6-1202.4E Outlet velocities shall be calculated. If outlet velocities equal or exceed erosive velocities of channel lining, then riprap or some other form of energy dissipation device shall be placed at the culvert outlet in accordance with § 6-1100 et. seq.

6-1203 Culvert Materials. Materials acceptable for use in culvert construction with the accompanying roughness coefficients are as set forth in § 6-0903.

6-1300 RETENTION, DETENTION, AND LOW IMPACT DEVELOPMENT FACILITIES

6-1301 General Requirements

6-1301.1 Stormwater retention and detention facilities are incorporated in the design of storm drainage systems to reduce the peak rate of discharge of the drainage system, reduce downstream erosion problems, possibly reduce the capital cost of the drainage system and help eliminate the environmental problems normally associated with the increased runoff of stormwater from new developments.

6-1301.2 Detention measures are extremely helpful for development in areas where downstream storm drainage systems are not adequate to receive the increased runoff being generated by the upstream development. These detention measures may be an adequate manner for meeting adequate offsite drainage requirements.

6-1301.3 Some methods for achieving stormwater detention are as follows:

6-1301.3A Rooftop storage

6-1301.3B Parking lot storage including both ponding and percolation trenches

6-1301.3C Retention and detention ponds

6-1301.3D Recreation area storage

6-1301.3E Road embankment storage

6-1301.3F Street and secondary drainage system storage during extreme intensity storms

6-1301.3G Porous asphalt pavement storage in parking areas.

6-1301.3H (33-90-PFM) Underground detention structures

6-1301.4 A few of these methods will be further explained in § 6-0000 et seq. with examples and design calculations.

6-1301.5 (46-94-PFM) The 2-yr, 2-hr and the 10-yr, 2-hr storm shall be the minimum used for the design of retention and detention facilities. A 2-yr, 24-hr and a 10-yr, 24-hr, SCS Type II storm (consistent with the hydrology specified in § 6-0801 and 6-0802) also may be used for design. Additionally, in the Four Mile Run Watershed, detention shall be provided for the 100-yr design storm.

6-1301.6 (46-94-PFM) Emergency spillways in ponds shall be designed to conform with § 6-1600 et seq., except that emergency spillways for ponds with watersheds less than 20 acres (8 ha) may be designed to discharge the 100-yr, 2-hr storm. Design of retention and detention facilities require the determination of actual volumes of rainfall occurring in a specific time and the actual volume of storm runoff in the same specified time. Routing of these volumes shall be incorporated in the design calculations.

6-1301.7 Other design parameters include the maximum allowable rate of runoff, characteristics of the developed area, and limitations of the developed area such as the maximum size of storage basin that can be incorporated in the topography, etc.

6-1302 Rooftop Storage

6-1302.1 Rooftop storage shall be designed to detain the 10-yr, 2-hr storm, and emergency overflow provisions must be adequate to discharge the 100-yr, 30-minute storm (see § 6-1302.5 and Tables 6.22 and 6.23).

6-1302.2 If a proper design is submitted for the 10-yr storm, sufficient storage will normally be provided for the 2-yr storm, and separate calculations need not be made.

6-1302.3 Rainfall from this design storm results in an accumulated storage depth of 3" (76mm).

6-1302.3A Because roof design in the County is currently based on a snow load of 30 PSF (1.44 KPA) or 5.8" (147mm) of water, properly designed roofs are structurally capable of holding 3" (76mm) of detained stormwater with a reasonable factor of safety.

6-1302.3B Roofs calculated to store depths greater than 3" (76mm) shall be required to show structural adequacy of the roof design.

6-1302.4 No less than 2 roof drains shall be installed in roof areas of 10,000 ft^2 (929 m^2) or less, and at least 4 drains in roof areas over 10,000 ft^2 (929 m^2) in area. Roof areas exceeding 40,000 ft^2 (3716 m^2) shall have 1 drain for each 10,000 ft^2 (929 m^2) area.

6-1302.5 Emergency overflow measures adequate to discharge the 100-yr, 30-minute storm must be provided.

6-1302.5A If parapet walls exceed 3" (76mm) in height, the designer shall provide openings (scuppers) in the parapet wall sufficient to discharge the design storm flow at a water level not exceeding 5" (125mm).

6-1302.5B One scupper shall be provided for every 20,000 ft^2 (1858 m^2) of roof area, and the invert of the scupper shall not be more than 3 1/2" (89mm) above the roof level. If such openings are not practical, then detention rings shall be sized accordingly.

6-1302.6 Detention rings shall be placed around all roof drains that do not have controlled flow.

6-1302.6A The number of holes or size of openings in the rings shall be computed based on the area of roof drained and runoff criteria.

6-1302.6B The minimum spacing of sets of holes is 2" (50mm) center-to-center.

6-1302.6C The height of the ring is determined by the roof slope and shall be 3" (76mm) maximum.

6-1302.6D The diameter of the rings shall be sized to accommodate the required openings and, if scuppers are not provided, to allow the 100-yr design storm to overtop the ring (overflow design is based on weir computations with the weir length equal to the circumference of the detention ring).

6-1302.6E Conductors and leaders shall also be sized to pass the expected flow from the 100-yr design storm.

6-1302.7 The maximum time of drawdown on the roof shall not exceed 17 hr.

6-1302.8 Josam Manufacturing Company and Zurn Industries, Inc. market "controlled-flow" roof drains. These products, or their equivalent, are accepted by the County.

6-1302.9 Computations required on plans:

6-1302.9A Roof area in ft² (m²)

6-1302.9B Storage provided at 3" (76mm) depth

6-1302.9C Maximum allowable discharge rate

6-1302.9D Inflow-outflow hydrograph analysis or acceptable charts. (For Josam Manufacturing Company and Zurn Industries, Inc. standard drains, the peak discharge rates as given in their charts are acceptable for drainage calculation purposes without requiring full inflow-outflow hydrograph analysis.)

6-1302.9E Number of drains required

6-1302.9F Sizing of openings required in detention rings

6-1302.9G Sizing of ring to accept openings and to pass 100-yr design storm.

6-1302.10 Example: Given – Building with flat roof 200' x 50', Predevelopment coefficient of runoff: c = 0.40, Predevelopment time of concentration t_c = 10 minute.

Computations:

6-1302.10A Roof Area = 200' x 50' = 10,000 ft²

6-1302.10B Storage provided at 3" of depth:

Vol.=(10,000 ft²) (3") (1/12) = 2,500 ft³

6-1302.10C Maximum allowable discharge (predevelopment rate of runoff)

Q = CIA = (0.4) (5.92) (927.2/.093) (1/43560)

Q = 0.54 CFS

6-1302.10D From Plate 37-6 1 set of holes with 3" of water will produce runoff or discharge of 6 GPM or 0.0134 CFS. See Plate 38-6 for a diagram of a typical ponding ring.

6-1302.10E Number of drains required for 10,000 ft² roof area equals 2.

6-1302.10F Sizing of openings:

Number of hole sets = allowable discharge divided by 0.0134 CFS/1 set of holes.

$$\text{Number of holes} = \frac{0.54 \text{ CFS/2 drains}}{0.0134 \text{ CFS/1 set of holes}}$$

20.1 sets of holes per drain (use 20 sets of holes).

6-1302.10G Size of ring:

Hole sets spaced 2" on center
Circumference = B x diameter
(20 sets) (2"/set) = B x diameter

D = 12.73", use 15" (see below if separate emergency overflow is not provided).

6-1302.11 If detention rings are to act as emergency overflow measures:

Q100=CIA t_c = 5 minutes
C = 1.0
A = 10,000 ft²/43,560 = 0.23 AC.

Q100=(1.0)(9.84)(0.23 AC.)=2.26 CFS

Weir formula: $Q = CLH^{3/2}$
C = 3.33
L = BD (circumference)
H = 2" or 0.17'

Assume all hole sets are clogged and the maximum allowable water depth on the roof is 5", or 2" above the 3" high ring.

$Q = CLH^{3/2}$
Q (per drain) = 2.26 CFS = 3.33 BD$(.17)^{3/2}$

D = 3.08' or 36.98" Use diameter of 37"

TABLE 6.22 RAINFALL DISTRIBUTION

Time Minutes	Total Precip In.	Total Precip Ft	Increm Precip In.	Increm Precip Ft
10-Year, 2-Hour Storm				
5	.60	.05	.60	.05
10	.99	.083	.39	.032
15	1.28	.107	.29	.024
20	1.52	.127	.24	.020
30	1.85	.154	.33	.027
40	2.11	.176	.26	.022
50	2.33	.194	.22	.018
60	2.50	.208	.17	.014
70	2.62	.218	.12	.010
80	2.72	.226	.10	.008
90	2.82	.235	.10	.008
100	2.89	.241	.07	.006
110	2.95	.246	.06	.005
120	3.00	.250	.05	.004
100-Year, 30-Minute Storm				
5	1.11	.093	1.11	.093
10	1.71	.143	.60	.050
15	2.16	.179	.45	.036
20	2.46	.204	.30	.025
30	3.00	.250	.54	.046

TABLE 6.23 STORM VOLUME IN INCHES OF RAINFALL

Duration of Storm

Frequency	30 Minute	1 Hr	2 Hr	3 Hr	6 Hr	12 Hr	24 Hr
1 Yr	1.0"	1.4"	1.7"	1.8"	2.1"	2.5"	2.7"
2 Yr	1.3"	1.8"	2.0"	2.1"	2.6"	3.0"	3.2"
5 Yr	1.7"	2.2"	2.6"	2.7"	3.2"	3.7"	4.5"
10 Yr	2.0"	2.6"	3.0"	3.2"	3.7"	4.6"	5.2"
25 Yr	2.3"	3.0"	3.5"	3.8"	4.2"	5.1"	6.0"
50 Yr	2.6"	3.4"	4.0"	4.4"	5.1"	6.0"	7.0"
100 Yr	3.0"	4.0"	4.5"	4.9"	5.4"	6.3"	7.3"
Max Prob					27.0"		

Average Relationship – 30 Minute Storm

5 Minutes - .37 of 30 Minutes
10 Minutes - .57 of 30 Minutes
15 Minutes - .72 of 30 Minutes

6-1303 Percolation Trenches. (See Plate 41A-6 (41AM-6))

6-1303.1 A percolation trench is designed to release stored runoff through percolation into the soil.

6-1303.2 Recharging stormwater back into the ground close to the point of rainfall is 1 of the more beneficial ways to treat stormwater runoff.

6-1303.2A Not only does it replenish the water table, it also eliminates the need for costly structures and improvements for conveying runoff from the developed site to an adequate outfall.

6-1303.2B The effects of recharging the water table must be analyzed before this method is used for detention.

6-1303.2C It is essential to determine if raising the water table will cause flooding or damage to nearby areas.

6-1303.3 (56-96-PFM) Percolation trenches may be useful only in areas where the soil is pervious and where the water table is lower than the design depth of the trench. A soils analysis prepared by a professional authorized by the State to provide such information must be submitted with design plans. The use of percolation trenches is undesirable in soil slippage areas.

6-1303.3A (47-95-PFM) Trenches shall be located so that percolation does not saturate soil within 4' (1.2m) of public roadway subgrades.

6-1303.4 Design of percolation trenches.

6-1303.4A The trench shall be designed to detain the 10-yr, 2-hr storm runoff; however, the 100-yr design storm should be routed through the system and adequate relief provided – usually in the form of overland relief.

6-1303.4B This method will also automatically provide the required detention for the 2-yr storm without additional calculations.

6-1303.4C VDOT #57 stone is recommended for filling the trench. This stone may be assumed to have 40% voids.

6-1303.4D Care must be taken, especially during construction, that sediment deposits do not clog the stone, thus preventing runoff from infiltrating into the stone.

6-1303.5 A trench utilizing stone with a high voids ratio may also be used in areas of impervious soil as temporary underground storage.

6-1303.5A Overland runoff enters the trench in the same manner, but release of runoff is controlled by an orifice, weir or other device at a rate not exceeding the predevelopment rate of runoff.

6-1303.5B The controlled runoff is usually released to an underground storm sewer system.

6-1303.6 Example of Percolation Trench Design:

6-1303.6A Given: Parking lot area 200' x 100', 90% impervious area (some planting), rate of infiltration of water into soil as determined by a soils analysis is 0.1"/minute.

Determine: The size of percolation trench required for 100% infiltration (no runoff).

Calculations:

6-1303.6A(1) Volume of runoff into the trench for 10-yr, 2-hr storm and 90% impervious surface:

6-1303.6A(1)(a) Total rainfall accumulation = 3"

6-1303.6A(1)(b) Vol.(in) = (200') (100') (0.9) (0.25') = 4,500 ft^3

6-1303.6A(2) Assume trench to be 200' long and 5' wide:

6-1303.6A(2)(a) Area of trench = 1,000 ft^2

6-1303.6A(2)(b) Vol.(out) = (1"/10 minutes) (60 minute/hr) (2 hr) (1'/12") (1,000 ft^2) = 1,000 ft^3

6-1303.6A(3) Vol.(stge.) = Vol.(in) – Vol.(out) = 3,500 ft^3 required storage for runoff (see table 6.24 and Plate 39-6).

6-1303.6A(3)(a) Using #57 stone at 40% voids: Vol. of stone = 3,500/0.40 = 8,750 ft^3

6-1303.6A(3)(b) Therefore, volume of trench = 8,750 ft³

6-1303.6A(4) Depth of trench = Volume/Area = 8,750 ft³/1,000 ft² = 8.75'

6-1303.6A(5) Rate of Discharge = Q(out) = (1"/10 minutes) (1'/12") (1 minute/60 sec.) (1,000 ft²) = 0.14 CFS

6-1303.6B As an alternate trench design, assume a trench area of 200' x 10'

6-1303.6B(1) Vol. (out) = 2,000 ft³ in 2 hr

6-1303.6B(2) Vol. (storage) = 4,500 – 2,000 = 2,500 ft³

6-1303.6B(3) Vol. of stone = 2,500/0.4 = 6,250 ft³

6-1303.6B(4) Depth of trench = 6,250/2,000 = 3.13'

6-1303.6B(5) Q(out) = 0.28 CFS

6-1303.7 An inflow-outflow hydrograph would provide a more accurate solution to the required storage.

6-1303.7A The rate of discharge from the trench (or infiltration into the soil) is determined by the area of the trench as well as soil characteristics.

6-1303.7B The rate of inflow to the trench (or rate of runoff from the parking lot) is determined by the hydrographs in Plates 40-6 (40M-6) and 41-6 (41M-6) and the design storm.

TABLE 6.24 MASS DIAGRAM ANALYSIS

Time	Rainfall Depth (ft)	LMP Area (ft²)	Vol.(in) (ft³)	Rate Out (CFS)	t (sec.)	Vol. (out)	Vol.(storage) = Vol.(in) – Vol.(out)
5	.050	18,000	900	0.14	300	42	858
10	.083	18,000	1494	0.14	600	84	1410
15	.107	18,000	1926	0.14	900	126	1800
20	.127	18,000	2286	0.14	1200	168	2118
30	.154	18,000	2772	0.14	1800	252	2520
40	.176	18,000	3168	0.14	2400	336	2832
50	.194	18,000	3492	0.14	3000	420	3072
60	.208	18,000	3744	0.14	3600	504	3240
70	.218	18,000	3924	0.14	4200	588	3336
80	.226	18,000	4068	0.14	4800	672	3396
90	.235	18,000	4230	0.14	5400	756	3474
100	.241	18,000	4338	0.14	6000	840	3498
110	.241	18,000	4338	0.14	6600	924	3504*
120	.250	18,000	4500	0.14	7200	1008	3492

* Maximum required storage is 3,504 ft³

6-1304 Pervious Pavement (98-07-PFM)

6-1304.1 Pervious pavement systems use a special asphaltic paving material (porous pavement) or open jointed concrete blocks (permeable pavement blocks) that allow stormwater to flow through the pavement or the open joints at a high rate. Water is temporarily retained below the pavement within an aggregate base and discharged to the storm sewer system or infiltrated into the underlying *in situ* soils. The principal components of pervious pavement systems are porous pavement or permeable pavement blocks, a bedding (choker) course, an optional filter fabric between the bedding course and the aggregate base in permeable pavement block systems, an open-graded aggregate base with a high void ratio, filter fabric to separate the aggregate base from the underlying soils and an underdrain that is connected to the storm drain system. Water quality control is provided by adsorption, filtering, sedimentation, biological action, and infiltration into the underlying soils. Pervious pavement systems reduce the peak rate and volume of stormwater runoff through detention storage and infiltration into underlying soils. Additional infiltration capacity or storage for detention can be obtained by increasing the depth of the aggregate base alone or in combination with storage chambers.

6-1304.1A Pervious pavement systems generally may be classified by the degree of infiltration into the underlying soils (i.e. exfiltration out of the aggregate base) that the systems are designed to achieve.

6-1304.1A(1) No Exfiltration. Systems that do not rely on infiltration of the captured stormwater runoff into the underlying soils are designed to provide water quality control and detention of storm water runoff from small storms. Water that has passed through the pervious pavement is discharged to the storm drain system through an unrestricted underdrain.

6-1304.1A(2) Full or Partial Exfiltration. Systems that provide for full or partial infiltration of the captured stormwater runoff into the underlying soils are designed to provide water quality control and retention of storm water. Such systems rely on infiltration to drain down the water stored in the aggregate base between storms. Pervious pavement systems designed for exfiltration, as utilized in Fairfax County, generally include underdrains that are capped or have restricted outflow. This allows the system to continue to provide water quality control and detention, albeit at reduced levels, if the infiltration capacity of the *in situ* soils is reduced over time due to consolidation of the soil bed or clogging of the soil pores.

6-1304.1B Pervious pavement systems are applicable as a substitute for conventional asphalt or concrete pavement. Pervious pavement systems require reasonably favorable conditions of land slope, subsoil drainage, and groundwater table. Pervious pavement systems are best suited to parking areas that are not subject to muddy conditions that cause sealing or clogging of the pervious material. Examples of suitable locations are parking areas for parks, churches, schools, office buildings, and shopping centers.

6-1304.1C For hydrologic computations using the Rational Method, the runoff coefficient (“C” factor) for porous pavement and permeable pavement block systems shall be computed based on the following formula:

$$C = (I - k_p) / I$$

Where:

I = design rainfall intensity (in/hr)
k_p = coefficient of permeability (in/hr)

Use a coefficient of permeability of 1.1 in/hr (27.9 mm/hr) for porous pavement and 3.0 in/hr (76.2 mm/hr) for permeable pavement block systems. For hydrologic computations using National Resource Conservation Service (NRCS) methods, use a Curve Number “CN” of 65 for porous pavement and 40 for permeable pavement block systems. For hydraulic computations, use a roughness coefficient (“n” value) of 0.01 for porous pavement and 0.03 for permeable pavement block systems.

6-1304.2 Location and Siting

6-1304.2A Pervious pavement systems may not be located in single family attached or detached residential developments for the purpose of satisfying the detention or water quality control (BMP) requirements of the Subdivision or Zoning Ordinance except as permitted under § 6-1304.2A(1) and § 6-1304.2A(2).

6-1304.2A(1) The Board of Supervisors (Board), in conjunction with the approval of a rezoning, proffered condition amendment, special exception, or special exception amendment, may approve the location of pervious pavement systems in single family attached or detached residential developments in accordance with the following criteria:

6-1304.2A(1)(a) Any decision by the Board shall take into consideration possible impacts on the environment and the burden placed on prospective owners for maintenance of the facilities;

6-1304.2A(1)(b) Pervious pavement must be part of an overall stormwater management design that does not rely exclusively on pervious pavement to meet water quality control (BMP) and detention requirements;

6-1304.2A(1)(c) Adequate funding for maintenance of the facilities shall be provided by the applicant where deemed appropriate by the Board;

6-1304.2A(1)(d) Pervious pavement facilities must be located on Home Owner Association (or "common") property and may not be located on individual buildable single family attached or detached residential lots, or any part thereof;

6-1304.2A(1)(e) Pervious pavement facilities shall be privately maintained and a private maintenance agreement in a form acceptable to the Director, which may include but is not limited to requirements for third-party inspections and the filing of annual maintenance and inspection reports with the County, must be executed before the construction plan is approved;

6-1304.2A(1)(f) The use of and responsibility for maintenance of pervious pavement facilities shall be disclosed as part of the chain of title to all future homeowners (e.g. individual members of a homeowners association) responsible for maintenance of the facilities; and

6-1304.2A(1)(g) In addition to the above requirements, reasonable and appropriate conditions may be imposed, where deemed appropriate by the Board, to provide for maintenance of the facilities and disclosure to property owners.

6-1304.2A(2) Pervious pavement systems may be located in single family attached or detached residential developments if the pervious pavement appears as a feature shown on a proffered development plan or a special exception plat approved prior to March 12, 2007.

6-1304.2B Pervious pavement systems may not be located on individual residential lots for the purpose of satisfying the water quality control (BMP) requirements of the Chesapeake Bay Preservation Ordinance.

6-1304.2C Pervious pavement systems that utilize infiltration may not be constructed on fill material.

6-1304.2D Pervious pavement systems may not be constructed in areas where the adjacent slopes are steeper than 20%.

6-1304.2E The slope of pervious pavement systems shall be from 1 to 5 percent.

6-1304.2F Setbacks. Pervious pavement systems not designed for infiltration into the underlying *in situ* soils shall be located a minimum of 10 feet (3 m) horizontally from building foundations preferably down gradient. Pervious pavement systems designed for infiltration into the underlying *in situ* soils shall be located a minimum of 20 feet (6 m) horizontally from building foundations preferably down gradient. Pervious pavement systems shall be located a minimum of 100 feet (30 m) horizontally from water supply wells. Pervious pavement systems not designed for infiltration shall be located a minimum of 25 feet (7.5 m) horizontally up gradient from septic fields and a minimum of 50 feet (15 m) horizontally down gradient from septic fields. Pervious pavement systems designed for infiltration shall be located a minimum of 50 feet (15 m) horizontally from septic fields preferably up gradient.

6-1304.2G The maximum flow length of impervious or pervious surfaces draining onto pervious pavement shall be 100 feet (30 m).

6-1304.2H The total drainage area to the pervious pavement shall not be greater than 5 acres (2.0 hectares).

6-1304.2H(1) The maximum ratio of impervious areas to the area of porous pavement for facilities designed to capture and treat a water quality volume of 0.5 inches (1.27 cm) is 3.4:1. The maximum ratio of impervious areas to the area of porous pavement for facilities designed to capture and treat a water quality volume of 1.0 inch (2.54 cm) is 1.2:1.

6-1304.2H(2) The maximum ratio of impervious areas to the area of permeable pavement blocks for facilities designed to capture and treat a water quality volume of 0.5 inches (1.27 cm) is 11:1. The maximum ratio of impervious areas to the area of permeable pavement blocks for facilities designed to capture and treat a water quality volume of 1.0 inch (2.54 cm) is 5:1.

6-1304.2I Pervious pavement systems shall not be located in the vicinity of loading docks, vehicle maintenance areas, or outdoor storage areas, where there is the potential for high concentrations of hydrocarbons, toxics, or heavy metals in stormwater runoff entering the facility.

6-1304.2J Pervious pavement systems shall not be located in travelways, areas subject to frequent truck traffic or material storage areas, such as loading docks, where there is potential for settling or high loads of grease and oils.

6-1304.2K Concentrated flow shall not be discharged directly onto pervious pavement.

6-1304.2L For pervious pavement systems utilizing open jointed concrete blocks, handicapped parking spaces and associated pathways shall utilize concrete blocks without open joints.

6-1304.3 Maintenance. Pervious pavement systems must be privately maintained and a private maintenance agreement must be executed before the construction plan is approved. The above does not preclude the use of pervious pavement by the County on County owned property. County maintained storm and sanitary sewer lines and their easements may be routed through areas of privately maintained pervious pavement.

6-1304.4 General Design Requirements.

6-1304.4A Water Quality Volume. For facilities designed to capture and treat the first 0.5 inches (1.27 cm) of runoff, the required water quality volume is 1,815 cubic feet per acre (127 m^3/ha) multiplied by the sum of the impervious area draining to the pervious pavement plus the area of the pervious pavement. For facilities designed to capture and treat the first 1.0 inch (2.54 cm) of runoff, the required water quality volume is 3,630 cubic feet per acre (254 m^3/ha) multiplied by the sum of the impervious area draining to the pervious pavement plus the area of the pervious pavement. The water quality volume must be filtered through the pavement to receive credit.

6-1304.4B Detention. For facilities designed to provide detention, the 2-year 2-hour storm and the 10-year 2-hour storm must be routed through the facility or the facility may be designed to infiltrate the 10-year 2-hour storm volume. Routings shall be performed in accordance with § 6-1300 *et seq*. Inlets shall be provided or the aggregate base extended 2 feet (0.6 meters) beyond the edge of the pavement to convey stormwater in excess of the water quality volume to the aggregate base or storage chambers below the pervious pavement.

6-1304.4C For facilities designed to provide detention, the maximum water surface elevation for the 10-year 2-hour storm shall be a minimum of 0.5 feet (152 mm) below the pavement bedding course.

6-1304.4D The detention release rate shall be controlled by a valve or cap on the end of the pavement underdrain within the structure.

6-1304.4E Pretreatment. Pretreatment for areas that sheet flow onto the pavement is not required. Inlets shall be designed to provide pretreatment of stormwater to prevent debris and sediments from entering the aggregate base or storage chambers. Where the aggregate base is extended beyond the edge of the pavement to convey stormwater to the aggregate base, an additional layer of filter fabric shall be provided 1 foot (305 mm) below the surface to prevent sediments from getting into the aggregate base.

6-1304.4F Underdrains shall be provided for all pervious pavement systems. The outfall of all underdrains must be in conformance with the adequate drainage requirements of § 6-0200 *et seq*.

6-1304.4G The bottom of the facility shall be a minimum of 4 feet (1220 mm) above the groundwater table and bedrock for facilities designed to provide infiltration and a minimum of 2 feet (610 mm) above the groundwater table and bedrock for all other facilities as determined by field run soil borings. The bottom of the facility shall be below the frost line to prevent frost heave of the pavement.

6-1304.4H For facilities designed to provide infiltration, the underdrain shall be restricted as necessary so that the design infiltration rate plus the underdrain outflow rate equals the design draw down rate. The restriction shall be achieved by using an end cap with a hole to act as an orifice or a valve fitted onto the end of the underdrain. Alternatively, a flow control satisfactory to the Director may be provided within the outflow structure. See § 6-1604.1A(2) for orifice calculations. The minimum diameter of any orifice shall be 0.5 inch (13 mm). Facilities shall be designed to dewater completely within 24 hours. If the facility can drain in the required time without any outflow through the underdrain, the end cap may be provided without a hole.

6-1304.4I For facilities utilizing infiltration, a soils analysis shall be prepared and infiltration tests conducted by a licensed professional engineer with experience in geotechnical engineering and soil evaluation, a certified professional soil scientist, or a certified professional geologist. Recommended guidelines for performing the field tests and soils analysis are available from the Department of Public Works and Environmental Services. A minimum field measured infiltration rate of 0.52 inches per hour (13.2 mm/hr) shall be required for infiltration. The design infiltration rate shall be half of the field measured rate. Soils with a CBR (minimum 96 hours soaked) less than 5 or that are highly expansive are not suitable for infiltration. Such soils would require compaction or other measures to be used as a pavement subgrade that would compromise their ability to infiltrate water. Pervious pavements on these soils shall be designed for no infiltration with unrestricted underdrains.

6-1304.4J Permeable pavement block systems require edge restraints to prevent movement of the pavement blocks from vehicle loads. Edge restraints may be standard VDOT curbs, standard VDOT combination curb and gutters, or precast or cast in place reinforced concrete borders a minimum 6 inches (152 mm) wide and 18 inches (457 mm) deep constructed with Class A3 concrete. Edge restraints shall be installed flush with the paver blocks.

6-1304.4K Side slopes of the facility excavated below ground may be as steep as the *in situ* soils will permit. The bottom of the excavated bed shall be level or nearly level. All excavation must be performed in accordance with Virginia Occupational Safety and Health (VOSH) requirements. If the facility is located on problem soils (such as marine clays), a geotechnical engineer shall specify the maximum acceptable slope for the excavation.

6-1304.4L Variations of the pervious pavement designs in Plates 78-6, 79-6, and 80-6 (78M-6, 79M-6, & 80M-6) may be approved by the Director provided the facility meets all of the requirements in § 6-1304 *et seq*.

6-1304.5 Pervious Pavement Design.

6-1304.5A Because there is no above ground storage of stormwater runoff, the minimum area of the pervious pavement required to infiltrate the water quality volume into the aggregate base is governed by the permeability of the pavement. The minimum area of the pervious pavement is computed as follows:

$$A_p = (WQ_v)/[(k_p/12)(t_s)]$$

Where:

A_p = area of pervious pavement (ft^2)
WQ_v = water quality volume (ft^3)
k_p = coefficient of permeability (in/hr)
t_s = time base of design storm (hrs)

6-1304.5B For design purposes, the permeability of the pavement is 1.1 in/hr (27.9 mm/hr) for porous pavement and 3.0 in/hr (76.2 mm/hr) for permeable pavement block systems and the time base of the design storm is 2 hours. After incorporating these values, the above equation reduces to:

A_p = 5.455 x WQ_v for porous pavement
A_p = 2.0 x WQ_v for permeable pavement block systems

6-1304.6 Aggregate Base/Storage Chamber Design.

6-1304.6A Storage Volume. Storage for detention or infiltration may be provided by a layer of aggregate or aggregate in combination with storage chambers beneath the pervious pavement. Water flows into the storage layer either through an inlet structure or through the pavement. Water flows out of the storage layer either by infiltration into the underlying *in situ* soils or through a restricted underdrain. The design objectives are to infiltrate as much of the water as possible, to assure that there is complete drain down of the facility between storms, to meet the structural requirements for the pavement design, and to meet the physical constraints of the site.

6-1304.6A(1) For facilities designed to infiltrate the water quality volume, the amount of storage required is based on water quality volume minus the infiltration rate into the underlying *in situ* soils and the outflow through the underdrain during the 2 hour filling period. The required storage volume is computed as follows:

$V_s = WQ_v – [(k_s)(A_s)(t_s)/12] – [3600(Q_u)(t_s)]$

Where:

V_s	= volume of storage (ft^3)
WQ_v	= water quality volume (ft^3)
k_s	= soil infiltration rate (in/hr)
A_s	= area of soil bed (ft^2)
t_s	= time base of design storm (hrs)
Q_u	= outflow through underdrain (cfs)

6-1304.6A(2) For facilities designed to provide detention in addition to treating the water quality volume, the water quality volume is replaced in the above equation by the total storm runoff volume for the design storm (V_{ds}). The required storage volume is computed as follows:

$V_s = V_{ds} - [(k_s)(A_s)(t_s)/12] – [3600(Q_u)(t_s)]$

6-1304.6B Storage Depth. Typically, the area of the soil bed will be known and the depth of the aggregate layer will be computed from the required storage and the porosity of the aggregate as follows:

For facilities designed to treat only the water quality volume:

$d_g = V_s/[(n_g)(A_s)]$

For facilities designed to provide detention add 0.5 feet (152 mm) to the above to provide the required separation (§ 6-1304.4C) between the bedding layer and the 10-year water surface elevation:

$d_g = V_s/[(n_g)(A_s)] + 0.5$

Where:

d_g	= depth of aggregate layer (ft)
V_s	= volume of storage (ft^3)
n_g	= porosity of aggregate
A_s	= area of soil bed (ft^2)

The depth of the aggregate layer does not include the thickness of the bedding layer.
For volume calculations, use a porosity of 0.40 for VDOT #2, #3, and #57 stone.

6-1304.6C Check the computed depth of the aggregate layer against the required depth for installation of the underdrain system [4 inches (102 mm) plus the diameter of the largest underdrain pipe] and the required depth of the pavement subbase (see § 7-0500 *et seq.*). The minimum required depth will be the greatest of these three values.

6-1304.6D Check the invert elevation of the aggregate layer against the elevation of the water table and bedrock. Also check that the facility can drain to the intended outfall.

6-1304.6E Facility Drain Time. The final step in the design of the aggregate layer is to compute the time that it takes the facility to drain. The facility must drain completely within 24 hours. The drain time is computed as follows:

$t_d = V_s/[(k_s)(A_s)/12 + 3600(Q_u)]$
Where:

t_d	= total drain time for facility (hrs)
V_s	= volume of storage (ft^3)
k_s	= soil infiltration rate (in/hr)
A_s	= area of soil bed (ft^2)
Q_u	= outflow through underdrain (cfs)

6-1304.6F For facilities designed with unrestricted underdrains, computation of the storage volume, storage depth, and facility drain time are not necessary. However, it is still necessary to check the depth of the aggregate layer against the required depth for the pavement sub-base and the invert elevation of the bottom of the aggregate layer against the elevation of the water table, bedrock, and the intended outfall.

6-1304.6G For facilities designed to provide infiltration, the infiltration rate into the underlying *in situ* soils typically will be less than the flow rate through the pavement and the outflow through the underdrain will be restricted or absent such that some storage will be required. In performing computations of the storage volume, storage depth, and facility drain time, initally assume that the underdrain is capped and there is no outflow through the underdrain. If the allowable depth of the storage layer based on the elevation of the groundwater table or bedrock is insufficient to provide the necessary storage volume, storage may be increased by increasing the area of the aggregate layer and soil bed or by incorporating storage chambers. Alternatively, the underdrain may be provided with an orifice to decrease the amount of storage needed. If the total drain time of the facility is in excess of 24 hours, it will be necessary to increase the area of the aggregate base and soil bed or provide an orifice and recompute the total drain time through the facility. Outflow through the orifice may not exceed the pre-development peak flow rates for the 2-year and 10-year storms.

6-1304.7 Underdrains. Underdrains shall consist of perforated pipe ≥ 4 inch (102 mm) in diameter placed in a layer of washed VDOT #57 stone. VDOT #2 or #3 stone may be substituted for #57 stone when #2 or #3 stone is used for the aggregate base. There shall be a minimum of 2 inches (51 mm) of aggregate above and below the pipe. Laterals shall be a minimum of 4-6 inches (102-152 mm) in diameter. Main collector lines and mainifolds shall be a minimum of 6-8 inches (152-203 mm) in diameter. Underdrains shall be laid at a minimum slope of 0.5%. Underdrains shall have a maximum internal spacing of 20 feet (6 m) on center and extend to within 10 feet (3 m) of the perimeter of the aggregate base. Underdrains not terminating in an observation well/cleanout shall be capped. Underdrain pipe connected to structures shall be nonperforated within 1 foot (305 mm) of the structure. Cleanouts and observation wells shall be nonperforated within 1 foot (305 mm) of the surface. All stone shall be washed with less than 1% passing a #200 sieve.

6-1304.8 Materials Specifications.

6-1304.8A Open jointed concrete blocks shall have a minimum thickness of 3 1/8 inches (80 mm) and conform to ASTM C 936-01 Standard Specification for Solid Concrete Interlocking Pavement Units. Joint openings shall be a minimum of 10% of the surface area of the pavement after installation. Joint openings shall be filled with VDOT #8, #8P, or #9 stone. VDOT #8 stone is recommended. VDOT #8P or #9 stone may be used where needed to fill narrow joints. All stone shall be washed with less than 1% passing a #200 sieve.

6-1304.8B Porous asphalt pavement shall be a minimum of 2.5 inches (64 mm) thick and conform to VDOT Road and Bridge Specifications for Asphalt Materials (Section 210) and Asphalt Cement (Section 211) except for aggregate gradation. The asphalt mix shall be 5.75% to 6.0% of dry aggregate by weight. The asphalt binder shall be modified with an elastomeric polymer to produce a binder meeting the requirements of PG 76-22 (AASHTO MP-1) and applied at a rate of 3.0% by total weight of the binder. Drain down of the asphalt binder shall be no greater than 0.3% (ASTM D 6390). The aggregate gradation shall be as specified in Table 6.28. Porous asphalt pavement shall have a minimum connected void space of 18%.

Table 6.28 Aggregate Gradation

U.S. Standard Sieve Size	Percent Passing
1/2 in (12.5 mm)	100
3/8 in (9.5 mm)	92-98
#4 (4.75 mm)	34-40
#8 (2.36 mm)	14-20
#16 (1.18 mm)	7-13
#30 (0.60 mm)	0-4
#200 (0.075 mm)	0-2

6-1304.8C The bedding course for open jointed pavement blocks shall consist of 1.5 to 3.0 inches (38-76 mm) of washed VDOT #8, #8P, or #9 stone. VDOT #8 stone is recommended. VDOT #8P or #9 stone may used to match the stone used in the joint openings. The thickness of the bedding course is to be based on the block manufacturer's recommendation. The bedding course for porous asphalt pavement shall consist of 1.0 to 2.0 inches (25-51 mm) of washed VDOT #57 stone. All stone shall be washed with less than 1% passing a #200 sieve.

6-1304.8D The aggregate base course shall consist of washed VDOT #57 stone. The thickness of the base course is determined by runoff storage needs, the infiltration rate of *in situ* soils, structural requirements of the pavement sub-base, depth to watertable and bedrock, and frost depth conditions. VDOT #2 or #3 stone may be substituted as the base course material provided an adequate choker course of VDOT #57 stone is provided between the aggregate base course and the bedding course. All stone shall be washed with less than 1% passing a #200 sieve.

6-1304.8E Underdrains shall be PVC pipe conforming to the requirements of ASTM F758, Type PS 28 or ASTM F949; HDPE pipe conforming to the requirements AASHTO M252 or M 294, Type S; or other approved rigid plastic pipe with a smooth interior. Underdrains shall be perforated with 4 rows of 3/8 inch (9.5 mm) holes with a hole spacing of 3.25 ± 0.25 inches (82.5 ± 6.4 mm) or a combination of hole size and spacing that provides a minimum inlet area ≥

1.76 square inches per linear foot (37.2 cm^2/m) of pipe or be perforated with slots 0.125 inches (3.2 mm) in width that provides a minimum inlet area $\geq$ 1.5 square inches per linear foot (31.8 cm^2 per linear meter) of pipe.

6-1304.8F Filter fabric. Filter fabric shall be a needled, non-woven, polypropylene geotextile meeting the requirements listed in Table 6.29. Heat-set or heat-calendared fabrics are not permitted.

Table 6.29 Filter Fabric Specifications

Grab Tensile Strength (ASTM D4632)	$\geq$ 120 lbs (534 N)
Mullen Burst Strength (ASTM D3786)	$\geq$ 225 lbs/in^2 (1550 kPa)
UV Resistance (ASTM D4355)	70% strength after 500 hours
Flow Rate (ASTM D4491)	$\geq$ 125 gal/min/ft^2 (5093 l/min/m^2)
Apparent Opening Size (AOS) (ASTM D4751)	US #70 or #80 sieve (0.212 or 0.180 mm)

6-1304.9 Construction Specifications.

6-1304.9A The owner shall provide for inspection during construction of the facility by a licensed professional (In accordance with standard practice, the actual inspections may be performed by an individual under responsible charge of the licensed professional). The licensed professional shall certify that the facility was constructed in accordance with the approved plans. The licensed professional's certification along with any material delivery tickets and certifications from the material suppliers and results of the inspections required under § 6-1304.9G(11) or § 6-1304.9H(6) and § 6-1304.9H(7) shall be submitted to the County prior to bond release. For projects requiring as-built plans, the required certification and supporting documents shall be submitted with or incorporated in the as-built plans. For projects that do not require as-built plans, the required certification and supporting documents shall be submitted with the RUP or non-RUP request.

6-1304.9B Pervious pavement facilities shall be constructed after the drainage area to the facility is completely stabilized. Erosion and sediment controls for construction of the facility shall be installed as specified in the erosion and sediment control plan. Preliminary grading of the area where pervious pavement is to be installed may be performed at the time the rest of the site is mass graded provided that positive drainage is maintained and the area is stabilized. For pervious pavement applications that will utilize infiltration, preliminary grading shall be a minimum of 2 feet (0.6 m) above the final design elevation of the bottom of the aggregate base and the area shall be immediately stabilized with no further construction traffic until the pervious pavement is installed.

6-1304.9C Areas where pervious pavement is to be installed should not be used for temporary sediment basins. Where unavoidable, the invert of the sediment basin shall be a minimum of 2 feet (0.6 m) above the final design elevation of the bottom of the aggregate base.

6-1304.9D For facilities designed for full or partial exfiltration, the floor of the facility shall be scarified to a minimum depth of 6 inches (152 mm) to reduce soil compaction and leveled before the filter fabric and stone are placed. Any areas of the facility where a temporary sediment basin was located also shall have 2 to 3 inches (51-76 mm) of sand incorporated into the *in situ* soils.

6-1304.9E Filter fabric shall be placed on the bottom and sides of the facility. Strips of fabric shall overlap by a minimum of 2 feet (0.6 m). Fabric shall be secured minimum of 4 feet (1.2 m) beyond the edge of the excavation. Following placement of the aggregate and again after placement of the pavement or pavers, the filter fabric should be folded over placements to protect installation from sediment inputs. Excess filter fabric should not be trimmed until the site is fully stabilized.

6-1304.9F After installation of the filter fabric over the soil subgrade, a 2 inch (51 mm) lift of aggregate shall be placed for the underdrain bedding. Underdrain piping shall be installed and sufficient aggregate shall be placed around and over the underdrain pipe to prevent damage to the pipe prior to compaction. Aggregate shall be placed in 4 to 8 inch (102-203 mm) lifts and compacted with a static roller. At least 4 passes should be made with a minimum 10-ton (9 T) static roller. The initial passes of the roller can be with vibration to consolidate the base material. The final passes should be without vibration. No visible movement should occur in the base material when compaction is complete.

6-1304.9G Installation of open jointed pavement blocks.

6-1304.9G(1) The bedding course shall be placed in a single lift. The bedding course shall be leveled and pressed (choked) into the aggregate base with at least 4 passes of a 10 ton (9 T) steel drum static roller. The bedding material should be moist to facilitate movement into the aggregate base. [Note: Install optional filter fabric per engineer's specifications prior to placement of bedding course.]

6-1304.9G(2) Edge restraints for open jointed pavement blocks shall be in place prior to installation of the bedding course and pavement blocks.

6-1304.9G(3) Prior to placement of the pavers, ¾ to 1 inch (19-25 mm) of the compacted bedding material shall be loosened and smoothed to an even surface.

6-1304.9G(4) Pavers may be placed by hand or with mechanical installers. Compact and seat pavers into the bedding material with a low amplitude 5000 lb-ft (22 kN), 75 to 95 Hz plate compactor.

6-1304.9G(5) Gaps at the edge of the paved areas shall be filled with cut pavers or edge units. When required, pavers shall be cut with a paver splitter or masonry saw. Cut pavers shall be no smaller than 1/3 of the full unit size.

6-1304.9G(6) Fill the openings and joints with aggregate until it is within ½ inch (13 mm) of the top surface. Remove excess aggregate by sweeping pavers clean. Compact the pavers again, vibrating the aggregate into the openings. Apply additional aggregate to the openings and joints, filling them completely. Remove excess aggregate by sweeping and compact the pavers. This will require at least 2 passes with the compactor.

6-1304.9G(7) Do not compact within 3 feet (0.9 m) of the unrestrained edges of the pavers.

6-1304.9G(8) The system must be thoroughly swept to remove any sediment or excess aggregate immediately after construction.

6-1304.9G(9) Proof roll the surface after installation is complete.

6-1304.9G(10) The area shall be inspected for settlement. Any blocks that settle shall be reset and reinspected.

6-1304.9G(11) The facility shall be inspected at 18-30 hours after a significant rainfall [0.5-1.0 inch (1.27-2.54 cm)] or artificial flooding to determine that the facility is draining properly.

6-1304.9H Installation of porous asphalt pavement.

6-1304.9H(1) The choker course shall be placed in a single lift. The choker course shall be leveled and pressed (choked) into the aggregate base with at least 4 passes of a 10 ton (9 T) steel drum static roller. The choker course material should be moist to facilitate movement into the aggregate base.

6-1304.9H(2) Porous asphalt pavement is installed similarly to regular asphalt pavement. The pavement shall be laid in a single lift over the choker course. The laying temperature shall be between 230°F and 260°F, with a minimum air temperature of 50°F, to make sure that the surface does not stiffen before compaction.

6-1304.9H(3) Compaction of the surface course should be completed when the surface is cool enough to resist a 10-ton roller. One or two passes of the roller are required for proper compaction. More rolling could cause a reduction in the porosity of the pavement.

6-1304.9H(4) The mixing plant shall certify to the aggregate mix, the abrasion loss factor, and the asphalt content in the mix. The asphalt mix shall be tested for its resistance to stripping by water using ASTM 1664. If the estimated coating area is not above 95%, additional antistripping agents shall be added to the mix.

6-1304.9H(5) The mix shall be transported to the site in a clean vehicle with smooth dump beds sprayed with a non-petroleum release agent. The mix shall be covered during transportation to control cooling.

6-1304.9H(6) The full permeability of the pavement surface shall be tested by application of clean water at a rate of at least 5 gpm (19 lpm) over the surface. All water must infiltrate directly without puddle formation or surface runoff.

6-1304.9H(7) The facility shall be inspected at 18-30 hours after a significant rainfall [0.5-1.0 inch (1.27-2.54 cm)] or artificial flooding to determine that the facility is draining properly.

6-1304.10 Plan Submission Requirements

6-1304.10A Plan view(s) with topography showing all hydraulic structures including underdrains.

6-1304.10B Cross section(s) of the facility with elevations showing the following as required: elevations and dimensions of inlet, outlet, underdrain, pavement course, bedding course, choker course, aggregate base, storage chambers, filter fabric, groundwater table, and bedrock.

6-1304.10C Sizing computations for the facility including volume of storage and surface area of the facility required and provided.

6-1304.10D Hydrologic calculations for the facility.

6-1304.10E Infiltration calculations as appropriate.

6-1304.10F Soils analysis and testing results for facilities that utilize infiltration including the elevation of the groundwater table and bedrock.

6-1304.10G A discussion of the outfalls from the facility is to be included in the outfall narrative.

6-1304.10H Construction and materials specifications.

6-1304.11 Pervious Pavement Design Example:

6-1304.11A Given:

Parking lot area = 20,000 ft^2;
Area of regular pavement (A_i) = 10,000 ft^2;
Area of porous asphalt pavement (A_p) = 10,000 ft^2;
Coefficient of permeability of porous asphalt pavement (k_p) = 1.1 in/hr
Design infiltration rate of *in situ* soils (k_s) = 0.26 in/hr (one-half of field measured rate of 0.52 in/hr);
Porosity of gravel (n_g) = 0.40

6-1304.11B Determine the required area of the porous asphalt pavement (A_p) for a water quality volume (WQ_v) of 1.0 inch per acre (3,630 ft^3) of impervious pavement plus 1.0 inch per acre (3,630 ft^3) of porous asphalt pavement. For design purposes, assume that the water quality volume can flow through the pervious pavement without surface runoff.

6-1304.11B(1) The water quality volume is:

$$WQ_v = 3{,}630\ ft^3\ (20{,}000\ ft^2 / 43{,}560\ ft^2) = 1{,}667\ ft^3$$

6-1304.11B(2) The required area of the porous asphalt pavement is:

$$A_p = 5.455 \times WQ_v = 5.455 \times 1667 = 9{,}094\ ft^2$$

Area provided 10,000 $ft^2 \geq 9{,}094\ ft^2$ OK

Note that as long as the ratio of impervious area to porous asphalt pavement meets the requirements of § 6-1304.2G [$\leq$ 3.4:1 for a water quality volume of 0.5 inches and $\leq$ 1.2:1 for a water quality volume of 1.0 inch], the area of the porous asphalt pavement will be sufficient to treat the water quality volume. In this example the ratio of impervious area to pervious pavement is 1:1.

6-1304.11C Determine the required storage volume (V_s) and depth (d_g) of the of the gravel layer to provide for infiltration of the entire water quality volume (WQ_v) [1,667 ft^3]. The design infiltration rate (k_s) is equal to half of the field measured rate of 0.52 in/hr. Assume that the area of the soil bed (A_s) is equal to the area of the porous asphalt pavement (A_p). Ignore any additional storage that may be provided by the underdrain pipes and assume that there is no outflow (Q_u) through the underdrain.

6-1304.11C(1) The required storage volume is:

$$\begin{aligned} V_s &= WQ_v - [(k_s)(A_s)(t_s)/12] - [3600(Q_u)(t_s)] \\ &= 1667 - [0.26(10000)(2)/12] - 0 \\ &= 1233.7\ ft^3 \end{aligned}$$

Use: $V_s = 1{,}234\ ft^3$

6-1304.11C(2) Compute the depth of the gravel storage area for a soil bed area of 10,000 ft^2 and a storage volume of 1,234 ft^3.

$$\begin{aligned} d_g &= V_s/[(n_g)(A_s)] \\ &= 1234/[(0.40)(10{,}000)] \\ &= 0.31\ ft \end{aligned}$$

6-1304.11C(3) Check the computed depth of the aggregate layer against the required depth for installation of the underdrain system and the required depth of the pavement subbase. The minimum required depth will be the greatest of these three values.

Check the depth between the bottom of the gravel storage area and the groundwater table and bedrock elevations from soil borings. Also check that the facility can drain to the intended outfall.

6-1304.11C(4) Compute the total drain time for the facility for a soil bed area of 10,000 ft^2 and a storage volume of 1,667 ft^3 (Must be less than 24 hrs.).

t_d $= V_s/[(k_s)(A_s)/12 + 3600(Q_u)]$
$= 1667/[(0.26)(10{,}000)/12 + 0]$
$= 7.7$ hrs ≤ 24 hrs OK

6-1304.11D Redesign the facility to provide detention of the 10-year 2-hour storm in addition to water quality control and to maximize infiltration. Note that an inlet or an extension of the aggregate base beyond the edge of the pavement will be required to deliver the storm volume in excess of the water quality volume (1,667 ft^3) to the gravel storage layer. The 10-year 2-hour storm volume is 3 inches per acre (10,890 ft^3) of impervious pavement and 3 inches per acre (10,890 ft^3) of porous asphalt pavement. Assume the gravel storage layer fills in 2 hours and that there is no outflow through the orifice during the filling period.

6-1304.11D(1) The 10-year 2-hour storm volume is:

V_{10} $= 10{,}890\ ft^3\ (20{,}000\ ft^2 / 43{,}560\ ft^2)$
$= 5{,}000\ ft^3$

6-1304.11D(2) Determine the required storage volume (V_s).

$V_s = V_{10} – [(k_s)(A_s)(t_s)/12] – [3600(Q_u)(t_s)]$
$= 5000 – [0.26(10{,}000)(2)/12] – 0$
$= 4{,}567\ ft^3$

6-1304.11D(3) Compute the depth of the gravel storage area for a soil bed area of 10,000 ft^2 and a storage volume of 4,567 ft^3.

d_g $= V_s/[(n_g)(A_s)] + 0.5$
$= 4567/[(0.40)(10{,}000)]$
$= 1.64$ ft

Check the depth between the bottom of the gravel storage area and the groundwater table and bedrock elevations from soil borings. If 1.64 ft is too deep, adjust the depth by providing additional storage in pipes or chambers or by enlarging the footprint of the facility.

6-1304.11D(4) Compute the total drain time for the facility for a soil bed area of 10,000 ft^2 and a storage volume of 4,567 ft^3 (Must be less than 24 hrs.).

t_d $= V_s/[(k_s)(A_s)/12 + 3600(Q_u)]$
$= 4567/[(0.26)(10{,}000)/12 + 0]$
$= 21.1$ hrs ≤ 24 hrs OK

6-1304.11E As a final example, assume that there is no infiltration into the *in situ* soils and we want to address an inadequate outfall by providing maximum detention of the 10-year storm. This will require that the entire 10-year storm volume be stored in the gravel storage layer and an orifice be designed for the underdrain system to keep the total drain time for the facility to less than 24 hrs.

6-1304.11E(1) Compute the depth of the gravel storage area for a soil bed area of 10,000 ft^2 and a storage volume of 5,000 ft^3.

d_g $= V_s/[(n_g)(A_s)] + 0.5$
$= 5000/[(0.40)(10{,}000)]$
$= 1.75$ ft

Check the depth between the bottom of the gravel storage area and the groundwater table and bedrock elevations from soil borings. If 1.75 ft is too deep, adjust the depth by providing additional storage in pipes or chambers or by enlarging the footprint of the facility.

6-1304.11E(2) Size an orifice to keep the total drain time for the facility to 24 hours for a maximum water level of 1.25 feet in the gravel storage layer (0.5 feet below the bedding layer) and a storage volume of 5,000 ft^3. The average energy head (H_o) above the centroid of the opening will be half of the maximum water level minus 2 inches (0.167 ft) for the pipe bedding.

The required discharge rate is computed from:

Q_u = $(V_{10}/t_d)/3600$
= (5000/24)/3600
= 0.0579 cfs

The size of the required orifice is computed using the standard orifice equation [see § 6-1604.1A(2)]:

$A = Q_o / C(2gH_o)^{1/2}$

Where:

Q_o = discharge (cfs)
C = orifice coefficient, typically set at 0.6 for sharp edged orifices but may vary depending on orifice geometry
A = flow area (ft²)
g = acceleration of gravity, 32.2 ft/sec²
H_o = energy head above centroid of opening (ft)

The average energy head is:

H_o = (1.25 – 0.167)/2
= 0.512

The orifice area is:

A = $0.0579/0.6(64.4*0.512)^{1/2}$
= 0.0168 ft²

The diameter of the orifice is:

D = $2(A/\pi)^{0.5}$
= $2(0.0168/3.1416)^{0.5}$
= 0.15 ft = 1.76 in

6-1304.11F Inflow-outflow hydrograph routings would provide a more accurate solution for these examples.

6-1305 Retention and Detention Ponds

6-1305.1 Small ponds created by constructing low earth dams across natural drainage courses or by excavating and regrading of a development site provide capacity for stormwater runoff detention.

6-1305.1A The ponds may be located in areas where other site development is expensive or unsuitable or may be made an integral part of the site landscaping designs.

6-1305.1B Stormwater permanently retained in these ponds may be considered a potential resource suitable for a variety of uses, including fire fighting, irrigation supplies and recreational sources.

6-1305.1C In addition to providing stormwater discharge reduction capabilities, detention ponds provide storage for sediment and pollution control in runoff, especially during the construction phase of development.

6-1305.1D (46-94-PFM) If embankments are used to dam natural drainage courses, they must be designed in accordance with § 6-1600 et seq.

6-1305.2 (46-94-PFM) Detention ponds and their primary outlet or principal spillway shall be designed to detain the increased runoff generated by development of a site based on the 2-yr and the 10-yr frequency design floods. Emergency or secondary spillways for detention ponds shall be designed in accordance with § 6-1600 et seq., except where the watershed is less than 20 acres (8 ha), in which case the Spillway Design Flood hydrograph may be obtained using the Rational Method.

6-1305.3 Outlets and emergency spillways shall be placed on either undisturbed ground or on a stabilized foundation and not in fill areas.

6-1305.4 (32-90-PFM) The planting of trees and other landscaping, except grass and other ground covers approved by the Director, on the structural embankment of any earth dam which intermittently or permanently impounds water, including stormwater management facilities, is prohibited.

6-1305.5 (32-90-PFM) All plans containing an earth dam which intermittently or permanently impounds water shall include a restrictive easement which covers the entire structural embankment and prohibits the planting of trees and all other landscaping, except grass and other ground covers approved by the Director. This easement shall be recorded in the Land Records of the County and shall run with the land.

6-1305.6 Design calculations for detention ponds shall be submitted with the site drainage plan and shall generally include the following:

6-1305.6A (46-94-PFM) Hydrographs of the 2-yr, 10-yr, emergency spillway and freeboard design storm inflow to the facility.

6-1305.6B Volume of storage vs. depth of storage curve.

6-1305.6C Outlet design calculations.

6-1305.6D Head discharge curve for the selected outlet size.

6-1305.6E (46-94-PFM) The routed or discharge hydrograph from the facility for the 2-yr, 10-yr, emergency spillway and freeboard design inflows.

6-1305.6F (46-94-PFM) Emergency spillway design calculations for ponds shall conform with the requirements of § 6-1603.

6-1305.6G (46-94-PFM) Embankment design computations shall conform with the requirements of § 6-1605.

6-1305.6H Calculations or effects (if any) on established floodplain boundaries.

6-1305.7 Other items that shall be included with or on the plans are:

6-1305.7A (46-94-PFM) When possible, the shape of the pond should conform with the natural topography.

6-1305.7B (46-94-PFM) Identification of required easements.

6-1305.7C Landscaping and fencing around detention ponds when access exposes the public to unusual risk.

6-1305.7D Properly executed maintenance agreements.

6-1305.7E (46-94-PFM) For wet detention ponds a drain valve shall be provided in accordance with § 6-1604.

6-1305.7F (35-91-PFM) The developer shall be required to provide and post signs informing the public where detention and retention ponds are to be located. Signs must be located so as to be visible from the adjoining lots and roadways from which the facilities may be viewed. The number of signs required for a site will depend upon the sight characteristics to meet the above visibility requirements. Signs shall be maintained by the developer from the time the plans for the ponds are approved by DPWES until bond release. At the time of bond release, the signs shall be removed by the developer. See Plate 41A-6 (41AM-6), for details.

6-1305.8 (46-94-PFM) Example of Detention Pond Design (for a detention pond with a watershed less than 20 acres):

6-1305.8A Given: A hypothetical 10-acre site to be developed into a commercial shopping center.

Design a detention pond in an existing natural drainage course. The natural topography limits the maximum height of the pond to 6', allowing 1' freeboard, the maximum height of water will be 5'.

6-1305.8B Calculations:

Predevelopment Runoff
Runoff coefficient: C = 0.30
Time of concentration: t_c = 15 minutes
Rainfall intensity:

I	=	3.5 in./hr for 2-yr frequency storm from Plate 3-6
I	=	5.1 in./hr for 10-yr frequency storm from Plate 3-6
Q_{2yr}	=	CIA = (0.30) (3.5 in./hr) (10 acre) = 10.5 CFS
Q_{10yr}	=	CIA = (0.30) (5.1 in./hr) (10 acre) = 15.3 CFS

Therefore, the maximum allowable runoff from the detention pond from the primary outlet(s) shall be limited to 10.5 CFS on the 2-yr storm and 15.3 CFS on the 10-yr storm.

Runoff after Development
Runoff coefficient: C = 0.9
Impervious area = (0.90) (10 acres) = 9 impervious acres. Time of concentration: t_c = 5 minutes

6-1305.8B(1) (46-94-PFM) From unit hydrograph (see Table 6.7) for 5 minute time of concentration, plot inflow hydrograph for a 9 impervious acre area (see Plate 42-6).

6-1305.8B(2) On the same graph, plot a straight line from the zero intercept to a point on the hydrograph at the 10.5 CFS point. The area between these 2 curves is the approximate volume of storage required. The planimetered area = 9.2 in.^{2} approximate volume = (9.20)(4,050) = 37,300 ft^3.

6-1305.8B(3) Limiting depth of storage is 5'. Assuming the 2-yr storm storage requirement will be approximately 75% of the 10-yr storage requirement, we have Vol. 10 yr/37,300 = 100/75 or 49,600 ft^3, say 50,000 ft^3, for 5' of depth. Hence, design detention pond to have a surface area of 10,000 ft^2.

6-1305.8B(4) Outflow pipe design – use FHA culvert charts in the VDOT Drainage Manual. Assume HW/D = 2.0 to 3.0

From concrete pipe culvert with inlet control chart at 10.5 CFS outflow: Diameter of culvert is 15" to 18". Assume 15" outlet pipe.

6-1305.8B(5) Plot the volume of storage vs. the depth of storage curve and the depth of storage vs. the discharge curve. The first curve is obtained from topography and grading data and the second curve is obtained from FHA culvert charts for the selected pipe size (see Plate 43-6).

6-1305.8B(6) Route the 2-yr inflow hydrograph through detention facility by using Plates 42-6 and 43-6. Following is a narrative of the procedure.

6-1305.9 The basic equation for determining the volume of storage required in a detention facility is that the volume of storage equals the volume of flow into the facility minus the volume of flow released from the facility (see established format in Table 6.25).

6-1305.9A For each 5 minute increment of time, the rate of flow into the facility is determined from the inflow hydrograph on Plate 42-6, is averaged with the rate of flow for the previous 5 minute increment, and this average value is multiplied by the time increment of 5 minutes or 300 seconds to obtain the incremental volume in for that particular 5 minute period (Column 3 of Table 6.25).

TABLE 6.25 – 2-YEAR STORM ROUTING

	1	2	3	4	5	6	7	8	9	10	11	12
Time	Inflow Rate CFS	Avg Inflow Rate CFS	Vol. ft³	Storage Carry-over	Total in ft³	Trial WS El.	Outflow Rate CFS	Avg Outflow Rate CFS	Vol Out ft³	Bal in Storage ft³	Corresp WS El.	
0	0		0				0					
		24.5						1.0				
5	49		7,350		7,350	.9'	2.0		300	7,050	.7	ok
		42						3.95				
10	35		12,600	7,050	19,650	1.8'	5.9		1,185	18,465	1.8	ok
		29.3						7.0				
15	23.5		8,780	18,465	27,245	2.6'	8.1		2,100	25,145	2.5	ok
		21.5						8.5				
20	19.5		6,440	25,145	31,585	3.0'	9.0		2,550	29,030	2.9	ok
		18						9.3				
25	16.5		5,400	29,030	34,340	3.3'	9.6		2,790	31,640	3.2	ok
		15.5						9.7				
30	14		4,650	31,640	36,290	3.4'	9.8		2,910	33,380	3.3	ok
		12.8						9.8				
35	11.5		3,840	33,380	37,220	3.4'	9.8		2,940	34,280	3.4	ok
		10.8						9.8				

40	10		3,240	34,280	37,520	3.4'	9.8		2,940	34,580	3.4	ok
		9.3						9.8				
45	8.5		2,800	34,580	37,380	3.4'	9.8		2,940	34,440	3.4	ok
		7.8						9.7				
50	7.0		2,340	34,440	36,780	3.3'	9.6		2,910	31,350	3.2	ok

Therefore, the maximum storage required is 34,580 ft³, maximum depth of storage is 3.4', and the maximum rate of discharge is 9.8 CFS.

6-1305.9B This incremental volume is summed with the balance in storage (Column 10 and Column 4) to yield the accumulated storage at the end of the particular time increment (Column 5).

6-1305.9C A trial water surface elevation is assumed and, from Plate 43-6, the rate of runoff or outflow is determined. These values are placed in Columns 6 and 7, respectively.

6-1305.9D An average outflow rate is calculated in Column 8, and multiplied by the time increment of 300 seconds to determine the volume released from the detention facility (Column 9).

6-1305.9E Volume (in) minus volume (out) equals storage (Column 10), and is compared to the trial water surface elevation assumed.

6-1305.9E(1) If the values differ by more than 0.1', assume a new trial water surface elevation and repeat the process.

6-1305.9E(2) If the values are less than 0.1' different, the routing for that 5 minute increment is balanced and the procedure is repeated for the next 5 minute increment, and so on.

6-1305.9F When the balance of runoff in storage (Column 10) begins to decrease in value, the detention pond is beginning to draw down and the maximum required volume of detention has been reached.

6-1305.9G The maximum rate of outflow, maximum required storage and maximum height of storage can be read directly from Table 6.25.

6-1305.9G(1) This particular problem yields the following results:

Q_{2yr} (max) = 9.8 CFS
Vol. of provided storage = 34,580 ft³
Max height of storage = 3.4'

6-1305.9G(2) These quantities compare favorably with the assumed 2-yr design of the pond and meet the 10.5 CFS maximum outflow requirements established above.

6-1305.10 Proceed now to the second stage of detention, which limits the 10-yr storm to the predeveloped peaks.

6-1305.10A Calculations:

6-1305.10A(1) Predevelopment Runoff
Runoff Coefficient: C=0.30
Time of Concentration: t_c = 15 minutes
Rainfall Intensity: I = 5.10 in./hr for 10-yr frequency storm from Plate 3-6
Q = CIA = (0.30) (5.10 in./hr) (10 acres) = 15.3 CFS

Therefore, the maximum allowable runoff from the detention pond from the primary outlet shall be limited to 15.3 CFS.

6-1305.10A(2) Runoff after Development
Runoff Coefficient: C = 0.9
Impervious Area = (0.90) (10 acres) = 9 acres
Time of Concentration: t_c =5 minutes

6-1305.10B From unit hydrograph for 5 minute time of concentration, plot inflow hydrograph for a 9 impervious acre area (see Plate 44-6).

6-1305.10C On the same graph, plot a straight horizontal line along the 5 CFS discharge rate which is an assumption that the average outflow rate from the lower pipe is half the peak rate found on the 2-yr routing above (i.e., 9.8/2 or say 5 CFS).

6-1305.10C(1) Since 34,580 ft³ of storage is required to fulfill the 2-yr detention requirement let us find out approximately at what time this volume would be fully utilized.

6-1305.10C(2) To approximate this time strike a vertical line at say 15 minutes and 25 minutes and measure the volumes between the inflow and assumed constant rate outflow. The values are 31,800 ft^3 and 44,000 ft^3 respectively.

6-1305.10C(3) This gives a difference of 12,200 ft^3 in 10 minutes or a rate of change of approximately 1,220/minute.

6-1305.10C(4) Since at 15 minutes we have used 31,800 ft^3 and we are looking for the time when 34,580 ft^3 will be required, we take 34,580 ft^3 - 31,800 ft^3 - 2,780 ft^3/1,220 ft^3/minute. = 2.2 minutes additional or say 17 minutes (15 minutes + 2.2 minutes).

6-1305.10C(5) Hence at 17 minutes the lower pond 2-yr detention facility is full; we draw a new discharge line from 17 minutes average flow of 5 CFS to the allowable discharge rate of 15.3 CFS on the inflow hydrograph curve and measure the volume between the inflow and outflow lines.

6-1305.10C(6) This gives a first approximation value of about 46,500 ft^3 or the amount of total storage required to accommodate both the 2-yr and 10-yr storms.

6-1305.10D Using the elevation – storage curve, we find the depth in the pond will be approximately 4.6'.

6-1305.10D(1) At this depth the 15" diameter pipe will discharge approximately 12.2 CFS.

6-1305.10D(2) With a permissible outflow of 15.3, this leaves a balance of 15.3 – 12.2 = 3.1 CFS to be provided by another opening.

6-1305.10D(3) Assuming a weir or slot of depth 4.6' – 3.4' = 1.2' head we can approximate the length from the weir formula $Q = 3LH^{3/2}$ or L = 0.8', say 1' long slot on a vertical riser beginning at elevation 3.4'.

6-1305.10E An elevation – discharge curve is developed for this slot and added to the 15" diameter pipe curve on Plate 43-6 beginning at elevation 3.4'.

6-1305.10F The routing is then performed as was done on the 2-yr storm and is shown on Table 6.26.

6-1305.10F(1) This results in a peak discharge out of 14.8 CFS at elevation 4.5'.

6-1305.10F(2) It reaches the peak at time 35 minutes and utilized 44,490 ft^3 of storage.

6-1305.10F(3) These quantities again compare favorably with the assumed 10-yr design parameters, and meet the 15.3 CFS maximum outflow requirements established above.

6-1305.10G (46-94-PFM) The emergency spillway shall be designed to meet the requirements of § 6-1603 and § 6-1604, except as noted in § 6-1305.2.

6-1305.10H When the depth of storage exceeds 4.5', outflow is expected to continue through the primary spillway as well as through the secondary emergency spillway.

6-1305.10H(1) (46-94-PFM) As an example, assume the outflow culvert pipe is clogged and the emergency spillway is a weir designed to pass the entire 100-yr storm:

Q(100) = CIA = (0.9) (9.84 in./hr) (10 acres) = 88.6 CFS
$Q\,(weir) = CLH^{1.5}$
C = 3.0
H = 0.9
Therefore, $L = (88.6) / (3.0)\,(0.9)^{1.5} = 34.6'$
6-1305.10H(2) The banks of the pond, the overflow weir, and the outlet channel must be adequately protected from erosion, and the capacity of the emergency overflow channel must be sufficient to pass the 100-yr storm. Energy dissipators will be required as appropriate.

TABLE 6.26 – 10-YEAR STORM ROUTING

	1	2	3	4	5	6	7	8	9	10	11	12
Time	In-flow Rate CFS	Avg Inflow Rate CFS	Vol. Ft³	Sto-rage Carry-over	Total in ft³	Trial WS El.	Out-flow Rate CFS	Avg Outflow Rate CFS	Vol Out ft³	Bal in Storage ft³	Cor-resp WS El.	
0	0		0				0					
		33						1.4				
5	66		9,900		9,900	.9'	2.3		420	9,480	.9	ok
		56						5.3				
10	47		16,300	9,480	26,280	2.5'	7.8		1,590	24,690	2.5	ok
		39.3						8.8				
15	31.5		11,775	24,690	36,465	3.4'	9.8		2,640	33,825	3.4	ok
		28.8						10.6				
20	26		8,625	33,825	42,450	3.8'	11.4		3,180	39,270	3.9	ok
		24						12.8				
25	22		7,200	39,270	46,470	4.9'	14.2		3,840	42,630	4.3	ok
		20.3						14.5				
30	18.5		5,550	42,630	48,180	4.4'	14.8		4,350	43,830	4.4	ok
		17						14.8				
35	15.5		5,100	43,830	48,930	4.4'	14.8		4,440	44,490	4.5	ok
		14.5						14.8				
40	13.5		4,350	44,490	48,840	4.4'	14.8		4,440	44,400	4.4	ok
		12.5						14.5				
45	11.5		3,750	44,400	48,150	4.3'	14.2		4,350	43,800	4.4	ok

Therefore, the maximum storage required is 44,490 ft³, maximum depth of storage is 4.5', and the maximum rate of discharge is 14.8 CFS.

6-1305.10I Kentucky 31 tall fescue, the standard grass for most planting, has limitations for use in detention ponds. If the roots are saturated for more than 3 days, the grass will die. If the drawdown time is less than 3 days, it can be used. For non-shaded locations, bermuda grass makes good cover, and is very water tolerant. It may be seeded or sprigged. If sprigged on 1' (300mm) centers in the spring or early summer prior to July 1, a good ground cover will develop in about 6 weeks. Reed canary grass is another satisfactory and water tolerant cover. It grows to a height of 3' to 4' (0.9m to 1.2m). It will grow in some shade but must be seeded in the fall from the current year's crop.

6-1305.10J (46-94-PFM) Table 6.7 and Plate 40-6 (40M-6) show inflow hydrographs for various 10-yr, 2-hr storms with times of concentration from 5 minutes to 30 minutes.

6-1306 Maintenance Design Considerations

6-1306.1 The maintenance impact of stormwater management and BMP facilities is considered to be a primary concern to the County and to the future operation of these facilities.

6-1306.2 Engineers in the preparation of plans for construction are urged to include maintenance and operation of these facilities as one of the primary design considerations.

6-1306.3 The following shall be included in design considerations:

6-1306.3A All access ways shall be designated on plans and cleared, graded, or constructed with the facility construction.

6-1306.3B Proximity of facilities to public right-of-way shall be considered in order to minimize the length of access way.

6-1306.3C Multiple accesses on major facilities should be provided.

6-1306.3D Standard drainage easement agreements are not acceptable for access; therefore, special access easement agreements are to be executed which shall preclude planting of shrubs, construction of fences and other structures within the easement.

6-1306.3E Grading of the access to and around facilities shall not create steep slopes (maximum 3:1), in order to accommodate easy access for maintenance vehicles.

6-1306.3F (46-94-PFM) All facilities, including wet ponds, underground chambers, etc., shall provide accessibility with an all weather vehicular access way with a minimum 12' (3.7m) wide surface. Surfaces may be made of geosystems such as Geogrid, Grassrings, Geoweb, or Grasscrete or may be made of asphalt, concrete or gravel. The specific situation and physical conditions shall be considered when choosing surface materials and access ways shall be designed to support the anticipated maintenance vehicles.

6-1306.3F(1) When a private pipestem driveway is used for maintenance access to a stormwater management facility, the pavement section shall be constructed in accordance with Plates 23-7 (23M-7) and 24-7 (24M-7). A CBR test is required for the shared portion of the pipestem driveway, and for CBR test values less than 10, 1" (25mm) of additional aggregate subbase shall be provided for each point below 10. Pavement sections based on Plate 23-7 (23M-7) may provide an equivalent thickness index through the use of thicker asphaltic concrete layers.

6-1306.3G As these facilities are generally in close proximity to dwellings and may be subject to vandalism, principal spillways and other devices shall be designed to minimize tampering.

6-1306.3H (72-01-PFM) Underground chambers shall provide 2 or more access points, at least one of which shall be a 4’ X 4’ (1.2m x 1.2m) access door, double leaf, aluminum, BILCO Model JD-2AL or approved equal, for each major storage chamber or run of pipe for ventilation and cleaning, and be large enough to accommodate cleaning equipment. Access doors installed in areas subject to vehicle loads shall be BILCO Model JD-2AL H 20 or approved equal. Generally, the minimum height where possible, shall be 72" (1.8m), in order to facilitate maintenance.

6-1306.3I (46-94-PFM) The design of the dry pond bottom shall include a concrete low flow channel (trickle ditch) in accordance with § 6-1604. The minimum pond floor grade shall be 2% minimum into the trickle ditch.

6-1306.3J (46-94-PFM) Trash racks for ponds shall be designed and provided in accordance with § 6-1604. Where trash racks are provided, they shall be removable as a unit by unbolting, without destroying the structure. Access to the trash rack shall be provided immediately above the rack in the underground chambers.

6-1306.3K Where pipe storage is permitted all pipes shall be reinforced concrete with parged joints in facilities maintained by DPWES. Any other material shall be specifically approved by the Director.

6-1306.3L Where utilized, underground chambers shall be only appurtenant structures to the site and through drainage storm sewer systems. The stormwater management facilities shall not be incorporated as an in-line system, but be designed as a parallel or perpendicular appurtenant structure.

6-1306.3M Underground chambers shall provide a smooth contoured bottom to facilitate silt and debris removal.

6-1307 Bioretention Filters and Basins (98-07-PFM)

6-1307.1 Bioretention filters and basins (a.k.a. rain gardens) are landscaped areas in shallow depressions that are subject to temporary ponding of stormwater runoff. The principal components of bioretention facilities are plants that tolerate fluctuations in soil moisture and temporary ponding of water, a mulch layer, an engineered soil media, a gravel layer, and an underdrain that is connected to the storm drain system or daylighted. The soil media is highly permeable and well drained. Water quality control is provided by filtering storm water runoff through the soil media and mulch, biological and chemical reactions in the soil, mulch, and root zone, plant uptake, and infiltration into the underlying soil. The void spaces in the soil can be used to store runoff for detention or infiltration to provide reductions in the peak rate and volume of stormwater runoff. Additional infiltration capacity or storage for detention can be obtained by using a gravel layer alone or in combination with storage chambers below the soil media.

6-1307.1A Bioretention filters are designed to provide water quality control and detention of storm water runoff from small storms. Bioretention filters include underdrains that allow water that has passed through the soil media to be freely discharged.

6-1307.1B Bioretention basins are designed to provide water quality control and retention of storm water. Bioretention basins rely on infiltration into the underlying in situ soils to drain down between storms. Bioretention basins, as utilized in Fairfax County, generally include underdrains that are capped or have restricted outflow. This allows a bioretention basin to be converted to a bioretention filter if the infiltration capacity of the in situ soils is reduced over time due to clogging of the soil pores.

6-1307.1C Bioretention facilities are best suited for small drainage areas that have low sediment loads. Pre-treatment techniques that allow runoff to flow from impervious surfaces through well established lawns, naturally vegetated buffers, or specially constructed filter strips are used to remove coarse and fine grained sediments that may otherwise clog the surface of facilities. Level spreaders or stone energy dissipaters may be used to prevent concentrated flow from creating scour paths within the facility. Bioretention facilities should not be located where wooded areas would not otherwise need to be cleared as part of the site development.

6-1307.1D (102-08-PFM) Trees within bioretention facilities may be used to meet the requirements of Chapter 122 of the Code and § 12-0000 et seq. of the PFM.

6-1307.2 Location and Siting.

6-1307.2A In residential areas, bioretention facilities and their appurtenant structures must be located on Home Owner Association (or “common”) property and may not be located on individual buildable single-family attached or detached residential lots or any part thereof for the purpose of satisfying the detention or water quality control (BMP) requirements of the Subdivision or Zoning Ordinance except as noted herein. The Director may approve the location of bioretention facilities on individual buildable single-family detached lots for subdivisions creating no more than 3 lots where it can be demonstrated that the requirement is not practical or desirable due to constraints imposed by the dimensions or topography of the property and where adequate provisions for maintenance are provided. Such approval by the Director shall be in writing and shall specify such conditions deemed necessary to ensure the effectiveness, reliability, and maintenance of the proposed facilities.

6-1307.2B Bioretention facilities may be located on individual single-family detached residential lots that are not part of a bonded subdivision to satisfy the BMP requirements of the Chesapeake Bay Preservation Ordinance for construction on the lot.

6-1307.2C Bioretention facilities that utilize infiltration may not be constructed on fill material.

6-1307.2D Bioretention facilities may not be constructed on slopes steeper than 15 percent.

6-1307.2E Setbacks. Bioretention filters shall be located a minimum of 10 feet (3 m) horizontally from building foundations preferably down gradient. Bioretention basins shall be located a minimum of 20 feet (6 m) horizontally from building foundations preferably down gradient. Bioretention facilities shall be located a minimum of 100 feet (30 m) horizontally from water supply wells. Bioretention filters shall be located a minimum of 25 feet (7.5 m) horizontally up gradient from septic fields and a minimum of 50 feet (15 m) horizontally down gradient from septic fields. Bioretention basins shall be located a minimum of 50 feet (15 m) horizontally from septic fields preferably up gradient. Bioretention facilities shall be set back a minimum of 2 feet (0.6 m) from property lines.

6-1307.2F Bioretention facilities shall not be located in the vicinity of loading docks, vehicle maintenance areas, or outdoor storage areas, where there is the potential for high concentrations of hydrocarbons, toxics, or heavy metals in stormwater runoff.

6-1307.2G The maximum drainage area to a bioretention filter shall be 2 acres (0.8 hectares). The maximum impervious area draining to a bioretention filter shall be 1 acre (0.4 hectares). The maximum drainage area to a bioretention basin shall be 1 acre (0.4 hectares). The maximum impervious area draining to a bioretention basin shall be 0.5 acres (0.2 hectares).

6-1307.2H No minimum size is specified for bioretention facilities to allow for application on sites with limited space or topographic constraints. Bioretention facilities should be “footprinted” into the available landscape to minimize land disturbance.

6-1307.2I Bioretention facilities may be designed as on-line or off-line facilities. Off-line facilities are preferred and are mandatory when any part of the inflow to the facility is from flow in a County storm drainage easement.

6-1307.3 Maintenance.

6-1307.3A Bioretention facilities and their appurtenant structures must be privately maintained and a private maintenance agreement must be executed before the construction plan is approved. Bioretention facilities may not be located in County storm drainage easements. The above does not preclude the use of bioretention facilities by the County on County owned property.

6-1307.3B Maintenance access must be provided for all bioretention facilities not located on individual buildable single family detached lots in accordance with § 6-1306. For bioretention facilities located on individual buildable single family detached lots, maintenance access shall be considered as an integral part of the design and designated on the plan.

6-1307.3C Bioretention facilities shall be posted with permanent signs designating the area as a water quality management area. Signs shall state that the facility is a water quality management area, water may pond after a storm, and the area is not to be disturbed except for required maintenance. Signs shall be posted at approximately 150 foot (46 m) intervals along the perimeter of the bioretention area with a minimum of one sign for each facility. See Plate 81-6 (81M-6).

6-1307.4 General Design Requirements.

6-1307.4A Water Quality Volume. For facilities designed to capture and treat the first 0.5 inches (1.27 cm) of runoff, the required water quality volume is 1,815 cubic feet per acre (127 m^3/ha) of impervious area. For facilities designed to capture and treat the first 1.0 inch (2.54 cm) of runoff, the required water quality volume is 3,630 cubic feet per acre (254 m^3/ha) of impervious area. The water quality volume must be captured and filtered through the system.

6-1307.4B Detention. For facilities designed to provide detention, the 2-year 2-hour storm and the 10-year 2-hour storm must be routed through the facility; or the facility may be designed to infiltrate the 10-year 2-hour storm volume; or the facility may be designed to filter the 10-year 2-hour storm volume. Except where the facility is designed to filter the 10-year 2-hour storm volume, a drop inlet with a trash rack or screen shall be provided to convey stormwater in excess of the water quality volume to a gravel layer or storage chambers below the soil media.

6-1307.4C For on-line facilities, the inlet must be designed to pass the peak flow rate for the 10-year storm. For off-line facilities, a flow splitter shall be used to capture the design storm (typically the water quality volume) and pass larger flows around the facility.

6-1307.4D Pre-Treatment. Pre-treatment shall be provided at all points of concentrated inflow to facilities. Pre-treatment generally consists of a vegetated filter strip or channel and an energy dissipation device. However, space constraints (e.g. parking lot islands) may limit the ability to provide a vegetated filter strip or channel. Where space permits, vegetated filter strips or channels shall be provided. Energy dissipation devices are required for all facilities at points of concentrated inflow. Where inflow is in the form of sheet flow, a vegetated filter strip shall be provided where space permits. Guidelines for sizing vegetated filter strips and channels are provided in Tables 6.30 and 6.31.

Table 6.30
Pretreatment Filter Strip Sizing

Inflow Surface	Impervious				Pervious			
Maximum Inflow Approach Length	35 ft (11 m)		75 ft (23 m)		75 ft (23 m)		150 ft (46 m)	
Filter Strip % Slope (6% max)	≤ 2	≥ 2	≤ 2	≥ 2	≤ 2	≥ 2	≤ 2	≥ 2
Minimum Filter Strip Length Feet (meters)	10 (3)	15 (5)	20 (6)	25 (8)	10 (3)	12 (4)	15 (5)	18 (6)

Table 6.31
Pretreatment Vegetated Channel Sizing*
(101-08-PFM)

% Impervious	≤ 33%		34% - 66%		≥ 67%	
Channel Slope (4% max)	≤ 2%	≥ 2%	≤ 2%	≥ 2%	≤ 2%	≥ 2%
Min. Length feet (meters)	25 (7.6)	40 (12.2)	30 (9.1)	45 (13.7)	35 (10.7)	50 (15.2)

* 1 acre (0.8 hectare) drainage area. 2 foot (0.6 m) wide channel bottom.

6-1307.4E The maximum surface storage depth from the top of the mulch layer to the elevation of the overflow weir or drop inlet shall be 1 foot (305 mm).

6-1307.4F Berms used to pond water in bioretention facilities shall be a maximum of 2.0 feet (610 mm) in height measured from the downstream toe-of-slope to the top of the berm. The width of the top of the berm shall be a minimum of 2.0 feet (610 mm). The side slopes of the berm shall be a maximum of 3:1. Berms and overflow weirs shall be sodded and pegged in accordance with the most recent edition of the Virginia Erosion and Sediment Control Handbook. Facilities with berms that are equal to or less than 2.0 feet (610 mm) in height or excavated facilities will not be subject to the requirements of § 6-1600 (Design and Construction of Dams and Impoundments).

6-1307.4G The side slopes of the facility above ground shall be a maximum of 3:1. Where space permits, gentle side slopes (e.g. 5:1) are encouraged to blend the facility into the surrounding landscape. Side slopes of the facility excavated below ground may be as steep as the in situ soils will permit. All excavation must be performed in accordance with Virginia Occupational Safety and Health (VOSH) requirements. If the facility is located on problem soils (such as marine clays), a professional engineer with experience in geotechnical engineering shall specify the maximum acceptable slope.

6-1307.4H An outlet structure must be provided to convey the peak flow for the 10-year storm. The outlet structure may be a drop inlet or weir. A minimum freeboard of 6 inches (152 mm) shall be provided from the maximum elevation of the 10-year storm to the top of the facility.

6-1307.4I An emergency overflow weir shall be provided for all facilities with berms. The emergency overflow weir must have the capacity to pass the peak flow from the 100-year storm without overtopping the facility. If the facility design includes a weir in the berm to convey the peak flow for the 10-year storm, it also may be designed to function as the emergency overflow weir. The minimum weir length shall be 2 feet (610 mm).

6-1307.4J The outfall of all outlet structures, emergency overflow weirs, and underdrains must be in conformance with the adequate drainage requirements of § 6-0200 et seq.

6-1307.4K Underdrains shall be provided for all bioretention filters and basins except that facilities on individual single-family detached residential lots that are not part of a bonded subdivision may be constructed without underdrains if the underdrain cannot be daylighted on the lot or connected to a storm sewer structure. If there are no underdrains, observation wells shall be installed to monitor drainage from the facility.

6-1307.4L The depth between the bottom of the facility and groundwater table or bedrock shall be a minimum of 4 feet (1220 mm) for bioretention basins and a minimum of 2 feet (610 mm) for bioretention filters as determined by field run soil borings.

6-1307.4M For facilities designed to provide infiltration, the underdrain shall be restricted as necessary so that the design infiltration rate plus the underdrain outflow rate equals the design draw down rate. The restriction shall be achieved by using an end cap with a hole to act as an orifice or a valve fitted onto the end of the underdrain. Alternatively a flow control satisfactory to the Director may be provided within the overflow structure. See § 6-1604.1A(2) for orifice calculations. The minimum diameter of any orifice shall be 0.5 inch (13 mm). Facilities shall be designed to dewater completely within 48 hours. If the facility can drain in the required time without any outflow through the underdrain, the end cap may be provided without a hole.

6-1307.4N The minimum soil media depth shall be 2.5 feet (762 mm). If large trees and shrubs are to be installed, soil depths shall be increased to a minimum of 4 feet (1219 mm). The bottom of the soil layer must be a minimum of 4 inches (102 mm) below the root ball of plants to be installed. A layer of 2-3 inches (51-76 mm) of mulch shall be placed on top of the soil media.

6-1307.4O For facilities utilizing infiltration, a soils analysis shall be prepared and infiltration tests conducted by a licensed professional engineer with experience in geotechnical engineering and soil evaluation, a certified professional soil scientist, or a certified professional geologist. Recommended guidelines for performing the field tests and soils analysis are available from the Department of Public Works and Environmental Services. A minimum field measured infiltration rate of 0.52 inches per hour (13.2 mm/hr) shall be required for infiltration. The design infiltration rate shall be half of the field measured rate.

6-1307.4P Variations of the bioretention filter and basin designs in Plates 82-6, 83-6, 84-6, 85-6, and 86-6 (82M-6, 83M-6, 84M-6, 85M-6, & 86M-6) may be approved by the Director provided the facility meets all of the requirements in § 6-1307 et seq.

6-1307.5 Filter Bed Design.

6-1307.5A The required surface area of the filter is based on the volume of water to be treated and the available storage in the ponding area computed as follows:

$A_f = WQ_v/h_f$

Where:

A_f = area of filter (ft^2)
WQ_v = water quality volume (ft^3)
h_f = maximum ponding depth (ft)

6-1307.5B The drain time through the filter is based on the volume of water to be treated and the hydraulic properties of the soil media in accordance with Darcy's law computed as follows:

$t_f = (WQ_v)(d_f)/[(k_f/12)(0.5h_f+d_f)A_f]$

Where:

t_f = drain time through filter (hrs)
WQ_v = water quality volume (ft^3)
d_f = depth of filter (ft)
k_f = coefficient of permeability (in/hr)
h_f = maximum ponding depth (ft)
A_f = area of filter (ft^2)

6-1307.5C A coefficient of permeability of 1.5 in/hr (38.1 mm/hr) for the soil media shall be used for sizing calculations. The water quality volume must drain through the filter section in 24 hours. In determining the drain time through the filter, assume that the rainfall event has ended and the ponding depth is at the maximum elevation prior to the initiation of drawdown.

6-1307.6 Gravel Layer/Storage Chamber Design.

6-1307.6A Storage Volume. Storage for detention or infiltration may be provided by a layer of gravel or gravel in combination with storage chambers beneath the soil media. Water flows into the storage layer either through an inlet structure or through the soil media layer. Water flows out of the storage layer either by infiltration into the underlying in situ soils or through a restricted underdrain. The design objectives are to infiltrate as much of the water as possible, to provide sufficient storage so that water can drain freely through the filter without being backed-up, to assure that there is complete drain down of the facility between storms, and to meet the physical constraints of the site.

6-1307.6A(1) For facilities designed to infiltrate the water quality volume, the amount of storage required is based on the flow rate through the filter minus the infiltration rate into the underlying in situ soils and the outflow through the underdrain during the filling period. The required storage volume is computed as follows:

$V_s = WQ_v – [(k_s)(A_s)(t_f)/12] – [3600(Q_u)(t_f)]$

Where:

V_s = volume of storage (ft^3)
WQ_v = water quality volume (ft^3)
k_s = soil infiltration rate (in/hr)
A_s = area of soil bed (ft^2)
t_f = drain time through filter (hrs)
Q_u = outflow through underdrain (cfs)

6-1307.6A(2) For facilities designed to provide detention in addition to filtering the water quality volume, the water quality volume is replaced in the above equation by the total storm runoff volume for the design storm (V_{ds}). The required storage volume is computed as follows:

$V_s = V_{ds} - [(k_s)(A_s)(t_f)/12] – [3600(Q_u)(t_f)]$

6-1307.6B Storage Depth. Typically, the area of the soil bed will be known (approximately equal to the area of filter bed for larger facilities) and the depth of the gravel layer will be computed from the required storage and the porosity of the gravel as follows:

$d_g = V_s/[(n_g)(A_s)]$

Where:

d_g = depth of gravel layer (ft)
V_s = volume of storage (ft^3)
n_g = porosity of gravel
A_s = area of soil bed (ft^2)

6-1307.6C After determining the depth of the gravel layer, check the invert elevation against the elevation of the water table and bedrock. Also check that the facility can drain to the intended outfall.

6-1307.6D Facility Drain Time. The final step in the design of the gravel layer is to compute the time that it takes the facility to drain. The facility must drain completely within 48 hours after the water quality volume has been captured by the filter section. The drain time is computed as follows:

$t_d = V_s/[(k_s)(A_s)/12 + 3600(Q_u)] + t_f$

Where:

t_d = total drain time for facility (hrs)
V_s = volume of storage (ft^3)
k_s = soil infiltration rate (in/hr)
A_s = area of soil bed (ft^2)
Q_u = outflow through underdrain (cfs)
t_f = drain time through filter (hrs)

6-1307.6E For facilities designed as bioretention filters with unrestricted underdrains, computation of the storage volume, storage depth, and facility drain time are not necessary. However, it is still necessary to check the invert elevation of the gravel layer against the elevation of the water table, bedrock, and the intended outfall.

6-1307.6F For facilities designed as bioretention basins, the infiltration rate into the underlying in situ soils typically will be less than the flow rate through the filter and the outflow through the underdrain will be restricted or absent such that some storage will be required. In performing computations of the storage volume, storage depth, and facility drain time, initially assume that the underdrain is capped and there is no outflow through the underdrain. If the allowable depth of the storage layer based on the elevation of the groundwater table or bedrock is insufficient to provide the necessary storage volume, storage may be increased by increasing the area of the filter and soil bed or by incorporating storage chambers. Alternatively, the underdrain may be provided with an orifice to decrease the amount of storage needed. If the total drain time of the facility is in excess of 48 hours, it will be necessary to provide an orifice and recompute the total drain time through the facility. Outflow through the orifice may not exceed the predevelopment peak flow rates for the 2-year and 10-year storms.

6-1307.6G A porosity of 0.40 for VDOT #57 stone shall be used for volume calculations.

6-1307.7 Underdrains. Underdrains shall consist of pipe $\geq$ 4 inch (102 mm) in diameter placed in a layer of washed VDOT #57 stone. There shall be a minimum of 2 inches (51 mm) of gravel above and below the pipe. Laterals shall be a minimum of 4-6 inches (102-152 mm) in diameter. Main collector lines and mainifolds shall be a minimum of 6-8 inches (152-203 mm) in diameter. Underdrains shall be laid at a minimum slope of 0.5%. Underdrains shall extend to within 10 feet (3 m) of the boundary of the facility and have a maximum internal spacing of 20 feet (6 m) on center. Underdrains shall be separated from the soil media by geotextile fabric or a 2-3 inch (51-76 mm) layer of washed VDOT #8 stone or 1/8–3/8 inch (3.2-9.5 mm) pea gravel. Underdrains not terminating in an observation well/clean-out shall be capped. The portion of underdrain piping beneath the planting soil bed must be perforated. All remaining underdrain piping, including cleanouts, must be non-perforated. All stone shall be washed with less than 1% passing a #200 sieve.

6-1307.8 Observation Wells and Cleanouts. There shall be a minimum of one observation well or cleanout per 1,000 square feet (93 m^2) of surface area. Observation wells and cleanouts shall be a minimum of 6 inches (152 mm) in diameter with a screw, or flange type cap to discourage vandalism and tampering extending above the BMP water surface elevation. Cleanouts shall be provided at the end of all pipe runs. Cleanouts and observation wells shall be solid pipe except for the portion below the planting soil bed which must be perforated. Observation wells that are not connected to underdrain piping shall be anchored to a footplate at the bottom of the facility.

6-1307.9 Materials Specifications.

6-1307.9A The bioretention soil media shall be composed of a mixture of 60-75% washed sand, 5-15% organic compost meeting the requirements of Table 6.32, and 10-35% topsoil. Topsoil shall be a sandy loam, loamy sand, silt loam or loam per USDA textural classification. The textural class of the topsoil shall be verified by a laboratory analysis. Topsoil shall be of uniform composition, containing no more than 8% clay, free of stones, stumps, brush, roots, or similar objects larger than 2 inches. Topsoil shall be free of Bermuda Grass, Quackgrass, Johnson Grass, Mugwort, Nutsedge, Poison Ivy, Canadian Thistle, Tearthumb, or other noxious weeds. Sand shall meet AASHTO M-6, ASTM C-33, or VDOT Section 202 Grade "A" Fine Aggregate specifications. Sand shall be clean and free of deleterious materials. The final soil mixture shall not contain any material or substance that may be harmful to plant growth, or a hindrance to plant growth or maintenance. The final soil mixture shall meet the requirements in Table 6.33. Each bioretention area shall have a minimum of one soil test performed on the final soil mixture. Test results and materials certifications shall be submitted to DPWES prior to bond release.

6-1307.9B Mulch shall be double shredded aged hardwood bark with a particle size greater than 0.5 inches (1.27 cm). Mulch shall be well aged, uniform in color, and free of salts, harmful chemicals, and extraneous material including soil, stones, and plant material. Well aged mulch is mulch that has been stockpiled or stored for 6-12 months.

6-1307.9C Underdrains shall be PVC pipe conforming to the requirements of ASTM F758, Type PS 28 or ASTM F949; HDPE pipe conforming to the requirements AASHTO M252 or M 294, Type S; or approved equivalent pipe. Underdrains shall be perforated with 4 rows of 3/8 inch (9.5 mm) holes with a hole spacing of 3.25 ± 0.25 inches (82.5 ± 6.4 mm) or a combination of hole size and spacing that provides a minimum inlet area ≥ 1.76 square inches per linear foot (37.2 cm^2/m) of pipe or be perforated with slots 0.125 inches (3.2 mm) in width that provides a minimum inlet area ≥ 1.5 square inches per linear foot (31.8 cm^2/m) of pipe.

6-1307.9D Filter fabric. Filter fabric shall be a needled, non-woven, polypropylene geotextile meeting the requirements listed in Table 6.34. Heat-set or heat-calendared fabrics are not permitted.

Table 6.32 Compost Specifications

pH	6.0-8.0
Soluble Salts (electrical conductivity)	<5 dS/m (mmhos/cm)
Nutrient Content (dry weight basis)	Nitrogen – 1% or above Phosphorus – 1% or above Potassium – 1% or above
Organic Matter Content (dry weight basis)	50-60%
Moisture Content (wet weight basis)	40-50%
Particle Size (aggregate size)	Pass through a $^1/_2$ inch screen or smaller
Maturity Indicator (percentage of control)	>80% of control
Stability (CO_2 evolution)	0-4 mg CO_2C per g OM per day
Trace Elements/Heavy Metals	Meet U.S. EPA Class A standard, 40 CFR § 503.13. Tables 1 and 3
Pathogens	Meet U.S. EPA Class A standard, 40 CFR § 503.32(a)

Table 6.33 Soil Media Specifications

pH	5.5-6.5
Total Organic Matter by Loss on Ignition (ASTM F1647, Method A)	≥ 1.5% (dry weight)
Soluble Salts	≤ 500 ppm

Table 6.34 Filter Fabric Specifications

Grab Tensile Strength (ASTM D4632)	≥ 120 lbs (534 N)
Mullen Burst Strength (ASTM D3786)	≥ 225 lbs/in^2 (1550 kPa)
UV Resistance (ASTM D4355)	70% strength after 500 hours
Flow Rate (ASTM D4491)	≥ 125 gal/min/ft^2 (5093 l/min/m^2)
Apparent Opening Size (AOS) (ASTM D4751)	US #70 or #80 sieve (0.212 or 0.180 mm)

6-1307.10 Bioretention Planting Plans.

6-1307.10A Bioretention planting plans and specifications shall be prepared by a certified landscape architect, horticulturist, or other qualified individual who has knowledge of the environmental tolerance, ecological functions, and ecological impacts of plant species. Planting plans shall be prepared in accordance with the requirements of § 12-0700.

6-1307.10B Depending on the bioretention planting plan type and application as detailed in § 6-1307.10G, a mixture of trees, shrubs, and perennial herbaceous plants with a high density of fibrous roots is required. Selected plants must be able to tolerate highly variable moisture conditions, generally dry with brief periods of inundation. Depending on site conditions, selected plants also must be able to tolerate exposure to wind and sun, as well as salt and toxins in runoff from roads, parking lots, and driveways. The use of native plant species is preferred. The acceptability of proposed plant materials will be determined by the Director. Guidance on the use and selection of plants for bioretention facilities is available from the Urban Forest Management Division.

6-1307.10C All plants shall conform to the latest version of American Standard for Nursery Stock published by the American Nursery and Landscape Association (ANSI Z60.1) for quality and sizing. Trees and shrubs shall be nursery grown unless otherwise approved and shall be healthy and vigorous, free from defects, decay, disfiguring roots, sun-scald, injuries, abrasions, diseases, insects pests, and all forms of infestations or objectionable disfigurements as determined by the Director.

6-1307.10D Trees shall be a minimum of 1 inch (25.4 mm) caliper. Shrubs shall be a minimum of 2 gallon (7.57 L) container size and herbaceous plants shall be a minimum of 6 inch (152 mm) diameter container size. Variations in size may be approved by the Director, based on the requirements of the specific plants listed in the schedule.

6-1307.10E The planting plan shall provide for plant community diversity and should consider aesthetics from plant form, color, and texture year-round. The bioretention facility design and selection of plant material shall serve to visually link the facility into the surrounding landscape. If trees and shrubs are part of the design, woody plant species shall not be placed directly within the inflow section of the bioretention facility.

6-1307.10F All plantings must be well established prior to release of the conservation deposit. Nursery stock trees and shrubs required by the approved plan shall be viable (healthy and capable of developing a trunk and branch structure typical for their species) at the time the conservation deposit is released.

6-1307.10G Bioretention Planting Plan Types and Applications.

6-1307.10G(1) Wooded planting plans. Wooded bioretention facilities are appropriate where the facility is located at wooded edges, in the rear of residential lots, or where a wooded buffer is required. Design guidelines include:

6-1307.10G(1)(a) A density of ten (10) trees per 1,000 square feet (93 m^2) of basin shall be used.

6-1307.10G(1)(b) A minimum of three species of trees and three species of shrubs shall be planted, with trees located on the perimeter to maximize shading of the bioretention area;

6-1307.10G(1)(c) Of the three species of trees, at a minimum one shall be a mid or understory species; 30-50% of the total quantity of trees planted shall be mid or understory trees;

6-1307.10G(1)(d) Two to three shrubs shall be planted for each tree (2:1 to 3:1 ratio of shrubs to trees);

6-1307.10G(1)(e) At least 3 species of perennial herbaceous ground cover shall be planted;

6-1307.10G(1)(f) Where the basin is planted at the specified density, interior and peripheral parking lot landscaping and tree cover credit will be granted if planting conforms to the requirements of Article 13 of the Zoning Ordinance and § 12-0702 and § 12-0703;

6-1307.10G(1)(g) Trees planted in wooded bioretention facilities may also fulfill the requirements of transitional screening if the planting conforms to the provisions of Article 13-300 of the Zoning Ordinance.

6-1307.10G(2) Ornamental garden planting plans. Ornamental garden bioretention facilities are appropriate on commercial sites, as a focal point within residential developments or located in the front yard of an individual residential lot. Design guidelines include:

6-1307.10G(2)(a)The facility should be considered as a mass planting bed with plants that have ornamental characteristics linking it to the surrounding landscape;

6-1307.10G(2)(b) The facility should contain a variety of plant species which will add interest to the facility with each changing season;

6-1307.10G(2)(c) A mixture of trees, shrubs and perennial herbaceous groundcover at an approximate ratio of 10% trees, 20% shrubs and 70% perennials shall be planted;

6-1307.10G(2)(d) When the size or location of the facility precludes the use of large shade trees, use of small ornamental trees shall be considered. Alternatively, a mixture of shrubs and perennials at an approximate ratio of 40% shrubs, 60% perennials may be used;

6-1307.10G(2)(e) Spacing of plant material is species specific and will be subject to review and approval of the Director. In general the facility shall be planted at a density that the vegetation will cover 80-90% of the facility after the second growing season.

6-1307.10G(3) Meadow garden planting plans. Meadow garden bioretention facilities lack woody material and are appropriate for small facilities, either on commercial or residential sites.
Design guidelines include:

6-1307.10G(3)(a) Plant material shall consists of a variety of grasses and wildflowers. Other groundcovers, rushes and sedges may be part of the mixture as well;

6-1307.10G(3)(b) Species of different heights, texture, as well as flowering succession shall be selected;

6-1307.10G(3)(c) Spacing of plant material is species specific and will be subject to review and approval of the Director. In general the facility shall be planted at a density that the perennial herbaceous vegetation will cover 80-90% of the facility after the second growing season.

6-1307.11 Construction Specifications.

6-1307.11A The owner shall provide for inspection during construction of the facility by a licensed design professional (In accordance with standard practice, the actual inspections may be performed by an individual under responsible charge of the licensed professional). The licensed professional shall certify that the facility was constructed in accordance with the approved plans. The licensed professional's certification along with any material delivery tickets and certifications from the material suppliers and results of the tests and inspections required under § 6-1307.9A, § 6-1307.11D, and § 6-1307.11K shall be submitted to the County prior to bond release. For projects requiring as-built plans, the required certification and supporting documents shall be submitted with or incorporated in the as-built plans. For projects that do not require as-built plans, the required certification and supporting documents shall be submitted with the RUP or non-RUP request.

6-1307.11B Bioretention facilities shall be constructed after the drainage area to the facility is completely stabilized. Erosion and sediment controls for construction of the facility shall be installed as specified in the erosion and sediment control plan.

6-1307.11C The components of the soil media shall be thoroughly mixed until a homogeneous mixture is obtained. It is preferable that the components of the soil media be mixed at a batch facility prior to delivery to the site. The soil media shall be moistened, as necessary, to prevent separation during installation.

6-1307.11D The soil media shall be tested for pH, organic matter, and soluble salts prior to installation. If the results of the tests indicate that the required specifications are not met, the soil represented by such tests shall be amended or corrected as required and retested until the soil meets the required specifications. If the pH is low, it may be raised by adding lime. If the pH is too high, it may be lowered by adding iron sulfate plus sulfur.

6-1307.11E For bioretention basins, the floor of the facility shall be scarified or tilled to reduce soil compaction and raked to level it before the filter fabric, stone, and soil media are placed.

6-1307.11F The soil media may be placed by mechanical methods with minimal compaction in order to maintain the porosity of the media. Spreading shall be by hand. The soil media shall be placed in 8-12 inch (203-305 mm) lifts with no machinery allowed over the soil media during or after construction. The soil media should be overfilled above the proposed surface elevation as needed to allow for natural settlement. Lifts may be lightly watered to encourage settlement. After the final lift is placed, the soil media shall be raked to level it, saturated, and allowed to settle for at least one week prior to installation of plant materials.

6-1307.11G Fill for the berm and overflow weir shall consist of clean material free of organic matter, rubbish, frozen soil, snow, ice, particles with sizes larger than 3 inches (76 mm), or other deleterious material. Fill shall be placed in 8-12 inch (203-305 mm) lifts and compacted to prevent settlement. Compaction equipment shall not be allowed within the facility on the soil bed. The top of the berm and the invert of the overflow weir shall be constructed level at the design elevation.

6-1307.11H Plant material shall be installed per § 12-0805.

6-1307.11I Planting shall take place after construction is completed and during the following periods: March 15 through June 15 and September 15 through November 15 unless otherwise approved by the Director.

6-1307.11J All areas surrounding the facility that are graded or denuded during construction of the facility and are to be planted with turf grass shall be sodded.

6-1307.11K The facility shall be inspected at 12-24 and 36-48 hours after a significant rainfall [0.5-1.0 inch (1.27-2.54 cm)] or artificial flooding to determine that the facility is draining properly. Results of the inspection shall be provided to DPWES prior to bond release.

6-1307.12 Plan Submission Requirements.

6-1307.12A Plan view(s) with topography at a contour interval of no more than one foot and spot elevations throughout the facility showing all hydraulic structures including underdrains.

6-1307.12B Cross section(s) of the facility showing the following: elevations and dimensions of berm, inlet, outlet, underdrain, soil media, underlying gravel layer, storage chambers, filter fabric, groundwater table, and bedrock.

6-1307.12C Plant schedule and planting plan specifying species, quantity of each species, stock size, type of root stock to be installed and amount of tree cover claimed for each tree species or spacing of shrubs and perennials within facility. Planting plan shall be in conformance with § 12-0700.

6-1307.12D Sizing computations for the facility including volume of storage and surface area of facility required and provided.

6-1307.12E Hydrologic calculations for the facility.

6-1307.12F Design calculations and specifications for all hydraulic structures including inlet structures, overflow weirs, and underdrain piping.

6-1307.12G Infiltration calculations as appropriate.

6-1307.12H Soils analysis and testing results for facilities that utilize infiltration. Elevation of groundwater table and/or bedrock.

6-1307.12I A discussion of the outfalls from the facility is to be included in the outfall narrative.

6-1307.12J Construction and materials specifications.

6-1307.13 Bioretention Design Example:

6-1307.13A Given:

Drainage area to the facility = 20,000 ft^2;
Impervious area (A_i) = 15,000 ft^2;
Depth of filter (d_f) = 2.5 ft
Maximum ponding depth (h_f) = 1.0 ft
Coefficient of permeability of filter bed (k_f) = 1.5 in/hr
Design infiltration rate of in situ soils (k_s) = 0.35 in/hr (one-half of field measured rate of 0.7 in/hr);
Porosity of gravel (n_g) = 0.40

6-1307.13B Determine the required area of the filter bed (A_f) for a water quality volume (WQ_v) of 1.0 inch per impervious acre (3,630 ft^3).

6-1307.13B(1) The water quality volume is:

$$WQ_v = 3{,}630 \text{ ft}^3 (15{,}000 \text{ ft}^2 / 43{,}560 \text{ ft}^2) = 1{,}250 \text{ ft}^3$$

6-1307.13B(2) The area of the filter bed is:

$$A_f = WQ_v/h_f = 1250/1.0 = 1{,}250 \text{ ft}^2$$

6-1307.13B(3) Compute the drain time through the filter for a filter area of 1,250 ft^2 (Must be less than 24 hrs.).

t_f $= (WQ_v)(d_f)/[(k_f/12)(0.5h_f+d_f)A_f]$
$= (1250)(2.5)/$
$[(1.5/12)(0.5(1.0)+2.5)1250]$
$= 6.67$ hrs $\leq$ 24 hrs OK

If the facility is to be designed as a bioretention filter, the sizing computations are complete and a standard underdrain will be installed with no flow restriction.

6-1307.13C Determine the required storage volume (V_s) and depth (d_g) of the of the gravel layer to provide for infiltration of the entire water quality volume (WQ_v) [1,250 ft^2]. The design infiltration rate (k_s) is equal to half of the field measured rate of 0.7 in/hr. Assume that the area of the soil bed (A_s) is equal to the area of the filter (A_f). Ignore any additional storage that may be provided by the underdrain pipes and assume that there is no outflow (Q_u) through the underdrain.

6-1307.13C(1) The required storage volume is:

V_s $= WQ_v - [(k_s)(A_s)(t_f)/12] - [3600(Q_u)(t_f)]$
$= 1250 - [0.35(1250)(6.67)/12] - 0$
$= 1{,}006.8$ ft^3

Use: $V_s = 1{,}007$ ft^3

6-1307.13C(2) Compute the depth of the gravel storage area for a soil bed area of 1,250 ft^2 and a storage volume of 1,007 ft^3.

d_g $= V_s/[(n_g)(A_s)]$
$= 1007/[(0.40)(1250)]$
$= 2.01$ ft

Use $d_g = 2.0$ ft

Check the depth between the bottom of the gravel storage area and the groundwater table and bedrock elevations from soil borings.

6-1307.13C(3) Compute the total drain time for the facility for a filter area and soil bed area of 1,250 ft^2, a storage volume of 1,007 ft^3, and a drain time through the filter of 6.67 hrs (Must be less than 48 hrs.).

t_d $= V_s/[(k_s)(A_s)/12 + 3600(Q_u)] + t_f$
$= 1007/[(0.35)(1250)/12 + 0] + 6.67$
$= 34.3$ hrs $\leq$ 48 hrs OK

If the soil infiltration rate is less than the coefficient of permeability of the filter and there is no outflow through the underdrain, the total drain time for the facility can also be computed from:

t_d $= WQ_v/[(k_s)(A_s)/12]$
$= 1250/[(0.35)(1250)/12]$
$= 34.3$ hrs

6-1307.13D Redesign the facility to provide detention of the 10-year 2-hour storm in addition to water quality control and to maximize infiltration. Note that a drop inlet will be required to deliver the storm volume in excess of the 1,250 ft^3 captured by the filter section to the gravel storage layer. The 10-year 2-hour storm volume is 3 inches (10,890 ft^3) per impervious acre. Assume the gravel storage layer fills in 2 hours and that there is no outflow through the orifice during the filling period.

6-1307.13D(1) The 10-year 2-hour storm volume is:

V_{10} $= 10{,}890$ ft^3 (15,000 ft^2 / 43,560 ft^2)
$= 3{,}750$ ft^3

6-1307.13D(2) Determine the required storage volume (V_s).

$V_s = V_{10} - [(k_s)(A_s)(t_f)/12] - [3600(Q_u)(t_f)]$
$= 3750 - [0.35(1250)(6.67)/12] - 0$
$= 3{,}506.8$ ft^2

6-1307.13D(3) Compute the depth of the gravel storage area for a soil bed area of 1,250 ft^2 and a storage volume of 2,963 ft^3.

d_g $= V_s/[(n_g)(A_s)]$
$= 3507/[(0.40)(1250)]$
$= 7.01$ ft

Check the depth between the bottom of the gravel storage area and the groundwater table and bedrock elevations from soil borings. If 7.01 ft is too deep, adjust the depth by providing additional storage in pipes or chambers or by enlarging the footprint of the facility.

6-1307.13D(4) Size an orifice for the underdrain system to keep the total drain time for the facility to less than 48 hrs.

The required discharge rate is computed from:

Q_u = [(V_{10}/t_d) – (k_s x A_s)/12] / 3600
= [(3750/48) – (0.35x1250)/12)] / 3600
= 0.0116 cfs

The size of the required orifice is computed using the standard orifice equation [see § 6-1604.1A(2)]:

$Q_o = CA(2gH_o)^{1/2}$

Where:

Q_o = discharge (cfs)
C = orifice coefficient, typically set at 0.6 for sharp edged orifices but may vary depending on orifice geometry
A = flow area (ft²)
g = acceleration of gravity, 32.2 ft/sec²
H_o = energy head above centroid of opening (ft)

There are two unknowns in this equation, the orifice area and the energy head. One approach is to assume an orifice size. Based on the assumed orifice size, the average energy head required for the design flow rate can be computed and compared to the depth of the gravel storage area. If the energy head is less than half the depth of the gravel storage area we can safely assume that the facility will drain in 48 hours.

Try the minimum size orifice (0.5 inch diameter; 0.001364 ft^2).

$H_o = [(Q_o/CA)^2]/2g$

$H_o = [(0.0116/0.6*0.001364)^2]/64.4$
= 3.12 ft $\geq$ 0.5 x 7.01 ft not OK

Try a 5/8 inch diameter orifice (0.002131 ft^2).

$H_o = [(Q_o/CA)^2]/2g$

$H_o = [(0.0116/0.6*0.002131)^2]/64.4$
= 1.28 ft $\leq$ 0.5 x 7.01 ft OK

6-1307.13E Inflow-outflow hydrograph routings would provide a more accurate solution for these examples.

6-1308 Vegetated Swales (98-07-PFM)

6-1308.1 Vegetated swales are open, shallow channels with vegetation covering the side slopes and bottom that collect and slowly convey stormwater runoff to downstream discharge points. The principal components of vegetated swales are a dense covering of plants, with a deep root system to resist scouring, that tolerate fluctuations in soil moisture and temporary ponding of water, check dams to pond water along the length of the swale, an engineered soil media, and an underdrain in a gravel layer that is connected to the storm drain system or daylighted. The soil media is highly permeable and well drained. Water quality control is provided by sedimentation, filtering of stormwater runoff through the vegetation and soil media, biological and chemical reactions in the soil and root zone, plant uptake, and infiltration into the underlying soils. Reductions in the peak rate of runoff are achieved due to increases in the time of concentration compared to conventional conveyance systems and the temporary storage provided by the check dams and the void spaces in the soil and underdrain gravel. Infiltration into the underlying soils may provide some volume reduction. Vegetated swales are best suited for small drainage areas that have low sediment loads.

6-1308.2 Location and Siting.

6-1308.2A In residential areas, vegetated swales and their appurtenant structures must be located on Home Owner Association (or "common") property and may not be located on individual buildable single-family attached or detached residential lots or any part thereof for the purpose of satisfying the detention or water quality control (BMP) requirements of the Subdivision or Zoning Ordinance except as noted herein. The Director may approve the location of vegetated swales on individual buildable single-family detached lots for subdivisions creating no more than 3 lots where it can be demonstrated that the requirement is not practical or desirable due to constraints imposed by the dimensions or topography of the property and where adequate provisions for maintenance are provided. Such approval by the Director shall be in writing and shall specify such conditions deemed necessary to ensure the effectiveness, reliability, and maintenance of the proposed facilities.

6-1308.2B Vegetated swales may be located on individual single-family detached residential lots that are not part of a bonded subdivision to satisfy the BMP requirements of the Chesapeake Bay Preservation Ordinance for construction on the lot.

6-1308.2C Vegetated swales may not be located in the VDOT right-of-way without specific approval from VDOT.

6-1308.2D Setbacks. Vegetated swales shall be located a minimum of 10 feet (3 m) horizontally from building foundations preferably downgradient. Vegetated swales shall be located a minimum of 100 feet (30 m) horizontally from water supply wells. Vegetated swales shall be located a minimum of 25 feet (7.5 m) horizontally up gradient from septic fields and 50 feet (15 m) horizontally down gradient from septic fields. Vegetated swales shall be set back a minimum of 2 feet (0.6 m) from property lines.

6-1308.2E Vegetated swales shall not be located in the vicinity of loading docks, vehicle maintenance areas, or outdoor storage areas, where there is the potential for high concentrations of hydrocarbons, toxics, or heavy metals in stormwater runoff.

6-1308.2F In order to maintain healthy growth, swales vegetated solely with grass shall be located so that they receive a minimum of 6 hours of sunlight daily during the summer months throughout the entire length of the swale.

6-1308.2G The maximum drainage area to a vegetated swale shall be 2 acres (0.8 hectares). The maximum impervious area draining to a vegetated swale shall be 1 acre (0.4 hectares).

6-1308.2H Vegetated swales typically are designed as on-line conveyance systems but may be used off-line as pre-treatment for other types of BMPs.

6-1308.3 Maintenance.

6-1308.3A Vegetated swales and their appurtenant structures must be privately maintained and a private maintenance agreement must be executed before the construction plan is approved. Vegetated swales may not be located in County storm drainage easements. The above does not preclude the use of vegetated swales by the County on County owned property.

6-1308.3B Maintenance access must be provided for all vegetated swales not located on individual buildable single family detached lots in accordance with § 6-1306 except that the access way may have a grass surface rather than an all weather surface. For vegetated swales located on individual buildable single family detached lots, maintenance access shall be considered as an integral part of the design and designated on the plan.

6-1308.3C Vegetated swales shall be posted with permanent signs designating the area as a water quality management area. Signs for vegetated swales with check dams (swales designed to capture and treat the water quality volume) shall state that the facility is a water quality management area, water may pond after a storm, and the area is not to be disturbed except for required maintenance. Signs for vegetated swales (grass) without check dams shall state that the facility is a water quality management area and that the grass is to be maintained at a 4-8 inch (10-20 cm) height. Signs shall be posted at approximately 150 foot (46 m) intervals along the length of the vegetated swale on alternating sides with a minimum of one sign for each swale. See Plate 81-6 (81M-6).

6-1308.4 General Design Requirements.

6-1308.4A Vegetated swales may be designed to capture and treat the water quality volume using check dams or as simple conveyance systems without check dams. Swales designed as simple conveyance systems are not as effective in reducing pollutants as swales designed to capture and treat the water quality volume. Swales designed as simple conveyance systems are more commonly vegetated with grass. Swales designed to capture and treat the water quality volume are more commonly vegetated similarly to bioretention facilities.

6-1308.4B Pre-treatment. Bioretention soil media is not cohesive and must be protected from erosive forces. Energy dissipation devices with level spreaders shall be provided at all points of concentrated inflow to vegetated swales.

6-1308.4C The hydraulic capacity of vegetated swales shall be calculated using the procedures found in §6-1000 of the PFM. For grass swales, an “n” value of 0.2 shall be used for flow depths up to 4 inches (102 mm) decreasing to 0.03 at a depth of 12 inches (305 mm). For swales vegetated with a combination of native grasses, other types of ground covers, and shrubs an “n” value of 0.15 shall be used.

6-1308.4D Vegetated swales shall be designed to convey the 10-year peak discharge within the channel and with a minimum freeboard of 6 inches (152 mm) at all check dams. The maximum velocity for the 2-year peak discharge shall be 3 feet (0.91 m) per second.

6-1308.4E Swales shall be trapezoidal in shape to provide an even distribution of flow along the channel bottom. The bottom width of swales shall be 2-10 feet (0.6-3.0 m). Side slopes shall be no steeper than 3:1. Swales may vary in width along their length to conform to site topography and design goals.

6-1308.4F The longitudinal slope of vegetated swales shall be 1-5 percent.

6-1308.4G Underdrains shall be provided for all vegetated swales.

6-1308.4H The depth between the bottom of the gravel underdrain and the groundwater table or bedrock shall be a minimum of 2 feet (610 mm) as determined by field run soil borings.

6-1308.4I The minimum soil media depth shall be 2.0 feet (610 mm) for vegetated swales designed to capture and treat the water quality volume (swales with check dams). If trees and large shrubs are to be installed, soil depths shall be increased to a minimum of 4 feet (1219 mm). The bottom of the soil layer must be a minimum of 4 inches (102 mm) below the root ball of plants to be installed. A layer of 2-3 inches (51-76 mm) of mulch shall be placed on top of the soil media in areas not planted with vegetation. Biodegradable erosion control netting conforming to Standard and Specification 3.36 of the Virginia Erosion and Sediment Control Handbook, 3rd edition, 1992, shall be used to retain the mulch and surface soils until the surface of the swale is established.

6-1308.4J The minimum soil media depth shall be 1.0 feet (305 mm) for vegetated swales (grass) designed to filter the water quality design flow (swales without check dams).

6-1308.4K The outfall of all vegetated swales and underdrains must be in conformance with the adequate drainage requirements of § 6-0200 et seq.

6-1308.4L Variations of the vegetated swale designs in Plates 87-6, 88-6, and 89-6 (87M-6, 88M-6, and 89M-6) may be approved by the Director provided the facility meets all of the requirements in § 6-1308 et seq.

6-1308.5 Water Quality Volume Based Design.

6-1308.5A For facilities designed to capture and treat the first 0.5 inches (1.27 cm) of runoff, the required water quality volume is 1,815 cubic feet per acre (127 m^3/ha) of impervious area. For facilities designed to capture and treat the first 1.0 inch (2.54 cm) of runoff, the required water quality volume is 3,630 cubic feet per acre (254 m^3/ha) of impervious area. The water quality volume must be ponded behind the check dams so that it can be filtered through the soil media.

6-1308.5B Check dams shall be provided along the length of the swale to provide storage of the water quality volume. The maximum height of check dams shall be 1.5 feet (457 mm). Check dams shall be located and sized such that the ponded water does not reach the toe of the next upstream check dam or create a tailwater condition on incoming pipes. The length of the channel segment over which water is ponded is a function of the slope of the swale and the height of the check dam computed as follows:

$L = h / s$

Where:
L = length of channel segment (ft)
h = height of check dam (ft)
s = channel slope (ft/ft)

Channel segment lengths for various combinations of check dam height and channel slope that may be used for preliminary design are listed in Table 6.35. In determining the minimum spacing between check dams, add 5 feet (1.5 m) to the computed channel segment length to assure that the ponded water does not reach the toe of the next upstream check dam.

Table 6.35
Channel Segment Length ft (m)

		Check Dam Height ft (cm)		
		0.5 (15)	1.0 (31)	1.5 (46)
Channel Slope %	1	50 (15.2)	100 (30.5)	150 (45.7)
	2	25 (7.1)	50 (15.2)	75 (22.9)
	3	16.7 (5.1)	33.3 (10.2)	50 (15.2)
	4	12.5 (3.8)	25 (7.1)	37.5 (11.4)
	5	10	20	30

		(3.0)	(6.1)	(9.1)

6-1308.5C The volume stored behind a check dam is the average channel cross-section area at the ponding elevation multiplied by the length of the channel reach subject to ponding. Because the channel cross-section area is zero at the head of the reach, the average cross-section area is one half of the channel cross-section area at the low point of the check dam. The storage volume provided behind an individual check dam is computed as follows:

$V_s = L \times 0.5A_s$

Where:

V_s = volume of storage (ft^3)
L = length of channel segment (ft)
A_s = cross-section area (ft^2) at the check dam

The channel cross-section area for a trapezoidal channel is computed as follows:

$A = by + Zy^2$

Where:

b = bottom width
y = flow depth
Z = side slope length per unit height (e.g., Z = 3 if side slopes are 3H:1V)

The channel cross-section area of a trapezoidal channel with 3:1 side slopes for various combinations of check dam height and bottom width that may be used for preliminary design are listed in Table 6.36.

Table 6.36
Channel Cross-section Area ft^2 (m^2)

		Check Dam Height ft (cm)		
		0.5 (15)	1.0 (31)	1.5 (46)
Bottom Width ft (m)	2 (0.6)	1.75 (0.16)	5.0 (0.46)	9.75 (0.91)
	3 (0.9)	2.25 (0.21)	6.0 (0.56)	11.25 (1.05)
	4 (1.2)	2.75 (0.26)	7 (0.65)	12.75 (1.18)
	5 (1.5)	3.25 (0.30)	8 (0.74)	14.25 (1.32)
	6 (1.8)	3.75 (0.35)	9 (0.84)	15.75 (1.46)
	7 (2.1)	4.25 (0.39)	10 (0.93)	17.25 (1.60)
	8 (2.4)	4.75 (0.44)	11 (1.02)	18.75 (1.74)
	9 (2.7)	5.25 (0.49)	12 (1.11)	20.25 (1.88)
	10 (3.0)	5.75 (0.53)	13 (1.21)	21.75 (2.02)

6-1308.6 Water Quality Design Flow Method.

6-1308.6A For grass swales that function primarily as conveyance systems, swale design for water quality treatment is based on the peak flow from a 2 inch (508 mm) 24-hour storm. The peak water quality flow should be increased along the swale length to reflect inflows. If a single design flow is used, the flow at the outlet shall be used.

6-1308.6B The peak water quality flow shall be conveyed at a maximum depth equal to or less than 3 inches (762 mm).

6-1308.6C The maximum velocity for the peak water quality flow shall be 1.0 ft/sec (0.3 m/sec). Flow velocity is computed using the continuity equation:

$V_{wq} = Q_{wq} / A_{wq}$

Where:

V_{wq} = design flow velocity (ft/sec)
Q_{wq} = design flow (cfs)
A_{wq} = cross-sectional area (ft^2) of flow at design depth

6-1308.6D The minimum hydraulic residence time (i.e. the time for water to travel the full length of the swale) shall be 18 minutes. The minimum hydraulic residence time may be reduced to 9 minutes if the majority of flow enters at the head of the swale. The swale length required to achieve a minimum hydraulic residence time of 18 minutes (1080 seconds) is:

$L = 1080V_{wq}$

Where:

L = minimum swale length (ft)
V_{wq} = design flow velocity (ft/sec)

6-1308.6E The minimum swale length for swales designed using the water quality design flow method shall be 100 feet (30.5 m). The minimum length may be achieved with multiple swale segments connected by culverts with energy dissipators.

1308.7 Underdrains. Underdrains shall consist of pipe $\geq$ 6 inch (152 mm) in diameter placed in a layer of washed VDOT #57 stone. There shall be a minimum of 2 inches (51 mm) of gravel above and below the pipe. The underdrain shall begin within 10 feet (3 m) of the upstream boundary of the swale. Underdrains shall be separated from the soil media by geotextile fabric or a 2-3 inch (51-76 mm) layer of washed VDOT #8 stone or 1/8–3/8 inch (3.2-9.5 mm) pea gravel. Underdrain pipe shall be perforated. All stone shall be washed with less than 1% passing a #200 sieve.

6-1308.8 Cleanouts. Cleanouts shall be placed every 100 feet (30.5 meters) along the length of the swale beginning at the upper end of the swale with a minimum of one cleanout per swale. Cleanouts shall be a minimum of 6 inches (152 mm) in diameter with a screw, or flange type cap to discourage vandalism and tampering. Cleanouts shall be nonperforated pipe except for the portion below the planting soil bed which must be perforated. For swales with check dams, the cap shall be above the BMP water surface elevation. For swales without check dams, the cap shall be above the ground surface.

6-1308.9 Materials Specifications.

6-1308.9A The bioretention soil media shall meet the requirements of § 6-1307.9A. Each vegetated swale shall have a minimum of one soil test performed on the final soil mixture. Test results and materials certifications shall be submitted to DPWES prior to bond release.

6-1308.9B Mulch shall meet the requirements of § 6-1307.9B.

6-1308.9C Underdrains shall meet the requirements of § 6-1307.9C.

6-1308.9D Filter fabric. Filter fabric shall meet the requirements of § 6-1307.9D.

6-1308.9E Check dams. Check dams may be constructed of non-erosive material such as wood, gabions, rip-rap, or concrete. Earthen berms or bio-logs also may be used to create check dams. Whatever material is used, check dams shall be designed to prevent erosion where the check dams intersect the channel side walls. Check dams shall be anchored into the swale wall a minimum of 2 feet (0.6 m) on each side with the toe protected by a suitable non-erodible material (e.g. stone). A notch or depression shall be placed in the top of the check dam to allow the 2-year flow to pass without coming into contact with the check dam abutments.

6-1308.10 Vegetated Swale Planting Plans.

6-1308.10A Planting plans are required for all vegetated swales planted with a mixture of shrubs, perennial herbaceous plants, and trees (optional). Planting plans are not required for vegetated swales only planted with grass.

6-1308.10B (102-08-PFM) Vegetated swale planting plans and specifications shall be prepared by a certified landscape architect, horticulturist, or other qualified individual who has knowledge of the environmental tolerance, ecological functions, and ecological impacts of plant species. Planting plans shall be prepared in accordance with the requirements of § 12-0514.

6-1308.10C A mixture of shrubs and perennial herbaceous plants with a high density of fibrous roots is required. The use of trees is optional. Selected plants must be able to tolerate highly variable moisture conditions, generally dry with brief periods of inundation, retard and withstand stormwater flows, and filter pollutants. Depending on site conditions, selected plants also must be able to tolerate exposure to wind and sun, as well as salt and toxins in runoff from roads, parking lots, and driveways. The use of native plant species is preferred. The acceptability of proposed plant materials will be determined by the Director. Guidance on the use and selection of plants for vegetated swales is available from the Urban Forest Management Division.

6-1308.10D Plant materials shall meet the requirements of § 6-1307.10C and § 6-1307.10D.

6-1308.10E The planting plan shall provide for plant community diversity and should consider aesthetics from plant form, color, and texture year-round. The vegetated swale design and selection of plant material shall serve to visually link the facility into the surrounding landscape. If trees and shrubs are part of the design, woody plant species shall not be placed directly within the inflow section of the swale.

6-1308.10F All plantings must be well established prior to release of the conservation deposit. Nursery stock trees and shrubs required by the approved plan shall be viable (healthy and capable of developing a trunk and branch structure typical for their species) at the time the conservation deposit is released.

6-1308.10G Design Guidelines for Vegetated Swale Planting Plans.

6-1308.10G(1) The facility should be considered as a mass planting bed with plants that have ornamental characteristics linking it to the surrounding landscape;

6-1308.10G(2) The facility should contain a variety of plant species which will add interest to the facility with each changing season;

6-1308.10G(3) A mixture of shrubs and perennial herbaceous groundcover at an approximate ratio of 25% shrubs and 75% perennials shall be planted;

6-1308.10G(4) If trees are part of the design, only small ornamental trees may be used (Category I & II per Table 12.7). Trees may be substituted for shrubs up to an approximate ratio of 10% trees, 20% shrubs, and 70% perennials;

6-1308.10G(5) The plants shall be placed along the bottom of the swale. The side slopes of the swale shall be fully stabilized with vegetation. Spacing of plant material is species specific and will be subject to review and approval of the Director. In general the facility shall be planted at a density that the vegetation will cover 80-90% of the facility after the second growing season.

6-1308.11 Grassed Swale Vegetation. A dense cover of water-tolerant, erosion-resistant grass must be established. The selection of an appropriate species or mixture of species is based on several factors including climate, soils, topography, and sun tolerance. Grasses used in swales shall have the following characteristics: a deep root system to resist scouring; a high stem density, with well-branched top growth; water-tolerance; resistance to being flattened by runoff; and an ability to recover growth following inundation. Swales shall be sodded and pegged to provide immediate stabilization of the swale.

6-1308.12 Construction Specifications.

6-1308.12A The owner shall provide for inspection during construction of the facility by a licensed professional (In accordance with standard practice, the actual inspections may be performed by an individual under responsible charge of the licensed professional). The licensed professional shall certify that the facility was constructed in accordance with the approved plans. The licensed professional's certification along with any material delivery tickets and certifications from the material suppliers and results of the tests and inspections required under § 6-1308.9A, § 6-1308.12D, and § 6-1308.12J shall be submitted to the County prior to bond release. For projects requiring as-built plans, the required certification and supporting documents shall be submitted with or incorporated in the as-built plans. For projects that do not require as-built plans, the required certification and supporting documents shall be submitted with the RUP or non-RUP request.

6-1308.12B Vegetated swales shall be constructed after the drainage area to the facility is completely stabilized. Erosion and sediment controls for construction of the facility shall be installed as specified in the erosion and sediment control plan.

6-1308.12C The components of the soil media shall be thoroughly mixed until a homogeneous mixture is obtained. It is preferable that the components of the soil media be mixed at a batch facility prior to deliver to the site. The soil media shall be moistened, as necessary, to prevent separation during installation.

6-1308.12D The soil media shall be tested for pH, organic matter, and soluble salts prior to installation. If the results of the tests indicate that the required specifications are not met, the soil represented by such tests shall be amended or corrected as required and retested until the soil meets the required specifications. If the pH is low, it may be raised by adding lime. If the pH is too high, it may be lowered by adding iron sulfate plus sulfur.

6-1308.12E The soil media may be placed by mechanical methods with minimal compaction in order to maintain the porosity of the media. Spreading shall be by hand. The soil media shall be placed in 8-12 inch (203-305 mm) lifts with no machinery allowed over the soil media during or after construction. The soil media should be overfilled above the proposed surface elevation as needed to allow for natural settlement. Lifts may be lightly watered to encourage settlement. After the final lift is placed, the soil media shall be raked to level it, saturated, and allowed to settle for at least one week prior to installation of plant materials.

6-1308.12F Fill for earthen check dams shall consist of clean material free of organic matter, rubbish, frozen soil, snow, ice, particles with sizes larger than 3 inches (76 mm), or other deleterious material. Fill shall be placed in 8-12 inch (203-305 mm) lifts and compacted to prevent settlement. Compaction equipment shall not be allowed within the facility on the soil bed. The top of the check dam shall be constructed level at the design elevation.

6-1308.12G Plant material shall be installed per § 12-0805.

6-1308.12H Planting shall take place after construction is completed and during the following periods: March 15 through June 15 and September 15 through November 15 unless otherwise approved by the Director.

6-1308.12I All areas surrounding the facility that are graded or denuded during construction of the facility and are to be planted with turf grass shall be sodded.

6-1308.12J Vegetated swales designed to capture and treat the water quality volume shall be inspected at 12-24 and 36-48 hours after a significant rainfall [0.5-1.0 inch (1.27-2.54 cm)] or artificial flooding to determine that the facility is draining properly. Results of the inspection shall be provided to DPWES prior to bond release.

6-1308.13 Plan Submission Requirements.

6-1308.13A Plan view(s) with topography at a contour interval of no more than one foot and spot elevations throughout the facility showing all hydraulic structures including underdrains.

6-1308.13B Typical cross section(s) of the swale showing the following: dimensions of swale, underdrain, soil media, underlying gravel layer, filter fabric, groundwater table, and bedrock. Cross section(s) of the check dams.

6-1308.13C Profile showing the following: invert of the swale, gravel underdrain and pipe, groundwater table, bedrock, and check dams.

6-1308.13D Detail(s) of check dams.

6-1308.13E Plant schedule and planting plan specifying species, quantity of each species, stock size, type of root stock to be installed and amount of tree cover claimed for each tree species or spacing of shrubs and perennials within facility. Planting plan shall be in conformance with § 12-0700.

6-1308.13F Sizing computations for the facility including volume of storage, channel cross-section, and spacing of check dams required and provided.

6-1308.13G Hydrologic and hydraulic calculations for the swale.

6-1308.13H Field run soil borings used to determine the elevation of the groundwater table and/or bedrock.

6-1308.13I A discussion of the outfalls from the facility is to be included in the outfall narrative.

6-1308.13J Construction and materials specifications.

6-1308.14 Vegetated Swale Water Quality Volume Design Example:

6-1308.14A Given:

Drainage area to the swale = 30,000 ft^2;
Impervious area (A_i) = 10,000 ft^2;
Slope of swale (s) = 2.5 %
Length of swale = 200 ft

6-1308.14B Determine the required check dam height and spacing and channel cross-section for a water quality volume (WQ_v) of 0.5 inch per impervious acre (1,815 ft^3).

6-1308.14B(1) The water quality volume is:

$$WQ_v = 1{,}815\ ft^3\ (10{,}000\ ft^2 / 43{,}560\ ft^2) = 417\ ft^3$$

6-1308.14B(2) Select height and spacing of check dams. For a channel slope of 2.5%, the channel segment lengths subject to ponding for check dam heights of 0.5, 1.0, and 1.5 feet are:

$L = h / s$
$= 0.5 \text{ ft} / 0.025 = 20 \text{ ft}$
$= 1.0 \text{ ft} / 0.025 = 40 \text{ ft}$
$= 1.5 \text{ ft} / 0.025 = 60 \text{ ft}$

To determine the minimum spacing between check dams, add 5 feet to the computed channel segment length to assure that the ponded water does not reach the toe of the next upstream check dam. To determine the number of check dams, divide the total channel length by the minimum spacing and round down to the nearest whole number. If the computed value is a whole number, subtract one. This gives us minimum spacings of 25, 45, and 65 feet for check dam heights of 0.5, 1.0, and 1.5 feet respectively.

The volume of water ponded behind an individual check dam is the required water quality volume divided by the number of check dams. For 7, 4, and 3 check dams, the required storage volumes are 59.6, 104.3, and 139 ft^3 respectively.

The minimum required cross-section areas are:

$A_x = 2.0(V_s / L)$
$= 2.0(59.6 \text{ ft}^3 / 20 \text{ ft}) = 5.96 \text{ ft}^2$
$= 2.0(104.3 \text{ ft}^3 / 40 \text{ ft}) = 5.22 \text{ ft}^2$
$= 2.0(139.0 \text{ ft}^3 / 60 \text{ ft}) = 4.64 \text{ ft}^2$

From Table 6.36, it can be seen that a trapezoidal channel with 0.5 foot high check dams would need to exceed the allowable channel bottom width to provide the required storage volume. A trapezoidal channel with a 3 foot bottom width and 1.0 foot high check dams would work as would a trapezoidal channel with a 2 foot bottom width and 1.5 foot high check dams. The results of the above computations are summarized in Table 6.37.

Table 6.37
Example Problem

	Check Dam Height		
	0.5	1.0	1.5
Number of Check Dams	7	4	3
Spacing of Check Dams	25 ft	45 ft	65 ft
Storage Volume (Vs)	59.6 ft^3	104.3 ft^3	139.0 ft^3
Cross-section Area (Required)	5.96 ft^2	5.22 ft^2	4.64 ft^2
Cross-section Area (Provided)	--	6.0 ft^2	9.75 ft^2
Channel bottom width	--	3 ft	2 ft

6-1308.15 Vegetated Swale Water Quality Flow Design Example.

6-1308.15A Given:

Drainage area to the swale = 30,000 ft^2;
Impervious area (A_i) = 10,000 ft^2;
Hydrologic Soil Group (HSG) of pervious area = "C"
Time of concentration T_c = 0.25 hr
Time of travel T_t = 0.15 hr per 100 ft
Slope of swale (s) = 2.5 %
Length of swale = 200 ft

Assume that flow enters uniformly along the length of the swale. Therefore, the required minimum hydraulic residence time required is 18 minutes (§ 6-1308.6D).

6-1308.15B Calculate the design flow for a 2 inch 24 hour storm using standard NRCS methods. The swale was modeled as 2 subareas and two 100 foot long reaches. T_c for the first 100 foot reach of the swale consists of 6 minutes for sheet flow to the swale and 9 minutes travel time in the swale. The resulting design flow is 0.54 cfs at the outlet of the swale. [Note that computation of the 2-year and 10-year storm flows should use a combined $T_c + T_t$ of 0.1 hr because of the lower "n" value and travel time in the swale at higher flow depths.]

6-1308.15C Flow in the swale is calculated based on Manning's equation for open channel flow.

$$Q = \frac{1.49}{n} A R^{0.67} s^{0.5}$$

Where:

Q = flow rate (cfs)
n = Manning's roughness coefficient (unitless)
A = cross-sectional area of flow (ft^2)

R = hydraulic radius (ft)
s = longitudinal slope (ft/ft)

For shallow flow depths in swales, channel side slopes may be ignored in the initial estimation of the bottom width. The following equation (a simplified form of Manning's equation) may be used to estimate the swale bottom width:

$b = Q_{wq}n_{wq} / 1.49y^{1.67}s^{0.5}$

Where:

b = bottom width of swale (ft)
Q_{wq} = water quality design flow (cfs)
n_{wq} = Mannings roughness coefficient for shallow flow conditions (unitless)
y = design flow depth (ft)
s = longitudinal slope (ft/ft)

6-1308.15D The design flow velocity is computed using the continuity equation.

$V_{wq} = Q_{wq} / A_{wq}$

Where:

V_{wq} = design flow velocity (ft/sec)
A_{wq} = $by + Zy^2$ = cross-sectional area (ft^2) for a trapezoidal cross-section
Z = side slope length per unit height (e.g., Z = 3 if side slopes are 3H:1V)

6-1308.15E Determine the maximum allowable velocity for a hydraulic residence time of 18 minutes.

V_{wq} = L/1080
= 200 ft/1080 sec
= 0.19 ft/sec

6-1308.15F Compute the estimated bottom width and design flow velocity using a flow depth of 3 inches (0.17 ft).

$b = 0.54(0.2)/1.49(0.25^{1.67})(0.025^{0.5})$
= 4.6 ft

$V_{wq} = 0.54 / (4.6*0.25 + 3*0.25^2)$
= 0.40 ft/sec

0.40 ft/sec > 0.19 ft/sec Not OK

6-1308.15G Recompute the estimated bottom width and design flow velocity using a flow depth of 2 inches (0.17 ft).

$b = 0.54(0.2)/1.49(0.17^{1.67})(0.025^{0.5})$
= 8.8 ft

$V_{wq} = 0.54 / (8.8*0.17 + 3*0.17^2)$
= 0.34 ft/sec

0.34 ft/sec > 0.19 ft/sec Not OK

6-1308.15H At this point it is clear that in order to achieve the 18 minute hydraulic residence time required the slope of the channel must be reduced or a longer channel constructed. We can do both simultaneously by constructing a channel with the same upstream and downstream invert elevations along a sinusoidal path. Try a 300 foot long channel. The resulting slope is 1.7%. The maximum allowable velocity for the channel would now be:

V_{wq} = L/1080
= 300 ft/1080 sec
= 0.29 ft/sec

6-1308.15I Recompute the estimated bottom width and design flow velocity using a flow depth of 2 inches (0.17 ft) and a slope of 1.7%.

$b = 0.54(0.2)/1.49(0.17^{1.67})(0.017^{0.5})$
= 10.7 ft

10.7 ft > 10 ft Not OK

$V_{wq} = 0.54 / (10.7*0.17 + 3*0.17^2)$
= 0.28 ft/sec

0.28 ft/sec < 0.29 ft/sec OK

6-1308.15J Although the velocity is acceptable, the swale bottom width is greater than the maximum allowable width. Repeat the procedure of lengthening the swale until an acceptable result is achieved.

6-1309 Tree Box Filters (98-07-PFM)

6-1309.1 A tree box filter is a type of bioretention filter contained in a precast or cast-in-place concrete structure. The principal components of a tree box filter are an inlet structure, a concrete box, a tree grate, plants that tolerate fluctuations in soil moisture and temporary ponding of water, a mulch layer, an engineered soil media, and an underdrain in a gravel layer that is connected to the storm drain system. The soil media is highly permeable and well drained. Water quality control is provided by filtering storm water runoff through the soil media and mulch, biological and chemical reactions in the soil, mulch, and root zone, and plant uptake.

6-1309.1A Tree box filters are best suited for small drainage areas that have low sediment loads such as parking lots, courtyards, and along privately maintained streets.

6-1309.1B (102-08-PFM) Trees in tree box filters may be used to meet the requirements of Chapter 122 of the Code and § 12-0000 et seq. of the PFM. Minimum planting area and minimum distance to barriers as required by § 12-0509.4E(5) must be met to use trees in tree box filters to meet tree cover requirements. Use of some small trees may be possible (Category I and II).

6-1309.2 Location and Siting.

6-1309.2A In residential areas, tree box filters and their appurtenant structures must be located on Home Owner Association (or "common") property and may not be located on individual buildable single-family attached or detached residential lots or any part thereof.

6-1309.2B Tree box filters may be located in the right-of-way subject to approval by VDOT.

6-1309.2C Tree box filters shall not be located in the vicinity of loading docks, vehicle maintenance areas, or outdoor storage areas, where there is the potential for high concentrations of hydrocarbons, toxics, or heavy metals in stormwater runoff unless effective pre-treatment is provided to reduce the concentrations.

6-1309.2D The maximum impervious area draining to a tree box filter shall be 0.25 acre (0.1 hectares).

6-1309.3 Maintenance.

6-1309.3A Tree box filters and their appurtenant structures must be privately maintained and a private maintenance agreement must be executed before the construction plan is approved. Tree box filters may not be located in County storm drainage easements. The above does not preclude the use of tree box filters by the County on County owned property.

6-1309.3B Maintenance access must be provided for all tree box filters. Access routes shall be depicted on plans for all facilities not located in parking lots or along streets.

6-1309.3C Tree box filters shall be stenciled (or a plaque provided) on the inside of the box in a location clearly visible upon removal of the tree grate designating the tree box as a water quality management facility. The stenciling or plaque shall state that the facility is a water quality management facility, water may pond after a storm, and the facility is not to be disturbed except for required maintenance.

6-1309.4 General Design Requirements.

6-1309.4A Water Quality Volume. For facilities designed to capture and treat the first 0.5 inches (1.27 cm) of runoff, the required water quality volume is 1,815 cubic feet per acre (127 m^3/ha) of impervious area. For facilities designed to capture and treat the first 1.0 inch (2.54 cm) of runoff, the required water quality volume is 3,630 cubic feet per acre (254 m^3/ha) of impervious area. The water quality volume must be captured and filtered through the system.

6-1309.4B Flow Rate Based Design. For facilities whose treatment capacity is based on a maximum flow rate, the design methodology shall be approved by the Director.

6-1309.4C Tree box filters shall be located adjacent to a storm drain inlet to capture runoff that bypasses the system during heavy rainfall events.

6-1309.4D The top of the structure shall include a grate that will allow vegetation to grow through it and that is capable of supporting H-20 loads. The grate shall be removable for maintenance.

6-1309.4E The inlet structure shall be a standard curb inlet meeting VDOT requirements. A stone energy dissipater or other approved method shall be provided at the end of the inlet throat running along the entire length of the inlet at the surface of the soil media.

6-1309.4F Tree boxes shall be constructed of precast or cast-in-place reinforced concrete meeting VDOT requirements for drainage structures.

6-1309.4G To reduce construction costs, the bottom of the box may be left open in areas where there is potential for infiltration.

6-1309.4H The maximum surface storage depth from the top of the mulch layer to the bottom of the grate shall be 1 foot (305 mm).

6-1309.4I An underdrain connected to the storm drain system shall be provided for all tree box filters. The outfall of all underdrains must be in conformance with the adequate drainage requirements of § 6-0200 et seq.

6-1309.4J The minimum soil media depth shall be 2.5 feet (762 mm). The bottom of the soil layer must be a minimum of 4 inches (102 mm) below the root ball of plants to be installed. A layer of 2-3 inches (51-76 mm) of mulch shall be placed on top of the soil media.

6-1309.4K Variations of the tree box filter design in Plate 90-6 (90-M6) may be approved by the Director provided the facility meets all of the requirements in § 6-1309 et seq.

6-1309.5 Filter Bed Design. The required surface area of the filter is based on the volume of water to be treated and the available storage in the ponding area computed as follows:

$A_f = WQ_v/h_f$

Where:

A_f = area of filter (ft^2)
WQ_v = water quality volume (ft^3)
h_f = maximum ponding depth (ft)

6-1309.6 Underdrains. Underdrains shall consist of perforated pipe $\geq$ 4 inch (102 mm) in diameter placed in a layer of washed VDOT #57 stone. There shall be a minimum of 2 inches (51 mm) of gravel above and below the pipe. Underdrains shall be laid at a minimum slope of 0.5%. Underdrains shall extend the length of the box from one wall to within 6 inches (152 mm) of the opposite wall and may be centered in the box or offset to one side. Underdrains shall be separated from the soil media by geotextile fabric or a 2-3 inch (51-76 mm) layer of washed VDOT #8 stone or 1/8–3/8 inch (3.2-9.5 mm) pea gravel. Underdrains shall include a cleanout with a locking cap that extends 6 inches (152 mm) above the soil media and is accessible by removing the grate. Cleanouts shall be nonperforated pipe equal to or greater in diameter than the underdrain pipe. All stone shall be washed with less than 1% passing a #200 sieve.

6-1309.7 Materials Specifications.

6-1309.7A The bioretention soil media shall meet the requirements of § 6-1307.9A. A minimum of one soil test shall be performed on the final soil mixture. Test results and materials certifications shall be submitted to DPWES prior to bond release.

1309.7B Mulch shall meet the requirements of § 6-1307.9B.

6-1309.7C Underdrains shall meet the requirements of § 6-1307.9C.

6-1309.7D Filter fabric. Filter fabric shall meet the requirements of § 6-1307.9D.

6-1309.8 Tree Box Filter Planting.

6-1309.8A A tree box filter shall be planted with a small tree or shrub that is able to tolerate highly variable moisture conditions, generally dry with brief periods of inundation. The selected plants must not have a root zone density or characteristics that will rapidly displace the soils or clog the underdrain. Depending on site conditions, selected plants also must be able to tolerate exposure to wind and sun, as well as salt and toxins in runoff from roads, parking lots, and driveways. The use of native plant species is preferred. The acceptability of proposed plant materials will be determined by the Director. Guidance on the use and selection of plants for tree box filters is available from the Urban Forest Management Division.

6-1309.8B Plant materials shall meet the requirements of § 6-1307.10C and § 6-1307.10D.

6-1309.8C All plantings must be well established prior to release of the conservation deposit. Nursery stock trees and shrubs required by the approved plan shall be viable (healthy and capable of developing a trunk and branch structure typical for their species) at the time the conservation deposit is released.

6-1309.9 Construction Specifications.

6-1309.9A The owner shall provide for inspection during construction of the facility by a licensed professional (In accordance with standard practice, the actual inspections may be performed by an individual under responsible charge of the licensed professional). The licensed professional shall certify that the facility was constructed in accordance with the approved plans. The licensed professional's certification along with any material delivery tickets and certifications from the material suppliers and results of the tests and inspections required under § 6-1309.7A, § 6-1309.9D, and § 6-1309.9H shall be submitted to the County prior to bond release. For projects requiring as-built plans, the required certification and supporting documents shall be submitted with or incorporated in the as-built plans. For projects that do not require as-built plans, the required certification and supporting documents shall be submitted with the RUP or non-RUP request.

6-1309.9B Tree box filters shall be constructed after the drainage area to the facility is completely stabilized. Erosion and sediment controls for construction of the facility shall be installed as specified in the erosion and sediment control plan. The concrete box may be installed with the other elements of the storm drainage collection system provided that it is flushed of any accumulated sediments prior to installation of the underdrain, filter fabric, and soil media components.

6-1309.9C The components of the soil media shall be thoroughly mixed until a homogeneous mixture is obtained. It is preferable that the components of the soil media be mixed at a batch facility prior to delivery to the site. The soil media shall be moistened, as necessary, to prevent separation during installation.

6-1309.9D The soil media shall be tested for pH, organic matter, and soluble salts prior to installation. If the results of the tests indicate that the required specifications are not met, the soil represented by such tests shall be amended or corrected as required and retested until the soil meets the required specifications. If the pH is low, it may be raised by adding lime. If the pH is too high, it may be lowered by adding iron sulfate plus sulfur.

6-1309.9E The soil media shall be placed by hand with minimal compaction in order to maintain the porosity of the media. Spreading shall be by hand. The soil media shall be placed in 8-12 inch (203-305 mm) lifts with no machinery allowed over the soil media during or after construction. The soil media should be overfilled above the proposed surface elevation as needed to allow for natural settlement. Lifts may be lightly watered to encourage settlement. After the final lift is placed, the soil media shall be raked to level it, saturated, and allowed to settle for at least one week prior to installation of plant materials.

6-1309.9F Plant material shall be installed per § 12-0805.

6-1309.9G Planting shall take place after construction is completed and during the following periods: March 15 through June 15 and September 15 through November 15 unless otherwise approved by the Director.

6-1309.9H The facility shall be inspected at 12-24 hours after a significant rainfall [0.5-1.0 inch (1.27-2.54 cm)] or artificial flooding to determine that the facility is draining properly. Results of the inspection shall be provided to DPWES prior to bond release.

6-1309.10 Plan Submission Requirements.

6-1309.10A Plan view(s) with topography at a contour interval of no more than one foot and spot elevations throughout the facility showing all hydraulic structures including underdrains.

6-1309.10B Cross section of the facility showing the following: elevations and dimensions of the structure, inlet, outlet, underdrain, soil media, and underlying gravel layer, and filter fabric.

6-1309.10C Plant schedule specifying species, quantity of each species, stock size, type of root stock to be installed and amount of tree cover claimed for each tree species. Planting schedule shall be in conformance with § 12-0701.5.

6-1309.10D Sizing computations for the facility including volume of storage and surface area of facility required and provided.

6-1309.10E Hydrologic calculations for the facility.

6-1309.10F Design calculations and specifications for all hydraulic structures including inlet structures and underdrain piping.

6-1309.10G A discussion of the outfalls from the facility is to be included in the outfall narrative.

6-1309.10H Construction and materials specifications.

6-1309.11 Tree Box Filter Design Example:

6-1309.11A Given:

Impervious area (A_i) draining to the facility = 1,500 ft^2;
Maximum ponding depth (h_f) = 1.0 ft

6-1309.11B Determine the required area of the filter bed (A_f) for a water quality volume (WQ_v) of 0.5 inch per impervious acre (1,815 ft^3).

6-1309.11C The water quality volume is:

$$WQ_v = 1{,}815\ ft^3\ (1{,}500\ ft^2 / 43{,}560\ ft^2) = 62.5\ ft^3$$

6-1309.11D The area of the filter bed is:

$$A_f = WQ_v/h_f = 62.5/1.0 = 62.5\ ft^2$$

6-1309.11E The minimum size tree filter box would be:

$$\sqrt{62.5} = 7.9\ ft$$

Use a square 8 ft X 8 ft box. A rectangular 6 ft X 11 ft box could also be used.

6-1310 Vegetated Roofs (98-07-PFM)

6-1310.1 A vegetated roof (a.k.a. green roof) is a roof system consisting of the structural components of the roof, a waterproof membrane, a drainage layer, a layer of growth media, and plants. Depending on the type of plants and the waterproof membrane specified, an irrigation system and a root barrier also may be provided. Vegetated roofs reduce the peak rate and volume of stormwater runoff through interception of rainfall and evapotranspiration. Vegetated roofs improve water quality by capturing and filtering airborne depositional pollutants and by plant uptake of dissolved pollutants. Additionally, a vegetated roof provides reductions in energy use for heating and cooling, improvements in air quality, and aesthetic benefits. Vegetated roofs are classified as extensive or intensive systems based on the depth of the growth media and function of the roof.

6-1310.1A Extensive systems are shallow systems, having a growth media depth of 3-6 inches (75-150 mm), a low unit weight, low construction cost, low plant diversity, and minimal maintenance requirements. Extensive systems are constructed when the primary purpose is to achieve environmental benefits and typically are only accessible for maintenance and inspection. Extensive systems may be constructed on slopes of up to 33%.

6-1310.1B Intensive systems have a growth media depth of 6 inches (150 mm) or greater, a greater unit weight, increased design sophistication and construction costs, increased plant diversity, greater water holding capacity, and increased maintenance requirements compared to extensive systems. Intensive systems often are accessible and provide an amenity for occupants of the building. Intensive systems may not be constructed on slopes greater than 10%.

6-1310.2 General Requirements.

6-1310.2A Vegetated roofs may be used on non-residential buildings (commercial, industrial, and institutional uses), parking structures, multi-family residential buildings including condominiums and apartments, and mixed-use buildings with a residential component.

6-1310.2B Vegetated roofs may not be used on single family detached or attached units for the purpose of satisfying the detention or water quality control (BMP) requirements of the Subdivision or Zoning Ordinance. Vegetated roofs may not be used on single family detached units in nonbonded subdivisions to satisfy the BMP requirements of the Chesapeake Bay Preservation Ordinance.

6-1310.2C Vegetated roofs must be privately maintained and a private maintenance agreement must be executed before the construction plan is approved.

6-1310.2D Post-development hydrology. For hydrologic computations using the Rational Method, the runoff coefficient “C” values for vegetated roofs in Table 6.6 shall be used. For hydrologic computations using Natural Resource Conservation Service (NRCS) methods, a curve Number “CN” value of 65 shall be used for intensive systems and a value of 70 shall be used for extensive systems. Other values may be approved by the Director, depending on the composition and depth of the growth media and the slope of the roof, upon submission of a hydrologic analysis of the water retention capacity of the system.

6-1310.3 Design of Vegetated Roofs.

6-1310.3A Extensive vegetated roof systems shall have a minimum growth media depth of 3 inches (75 mm) and a maximum growth media depth of 6 inches (150 mm). The Director may approve growth media depths less than 3 inches for systems constructed on existing buildings when necessary because the structural design of the roof is not sufficient to carry the greater loads. Adjustments to the assigned runoff coefficients and curve numbers will be necessary to account for the reduced water holding capacity of the growth media.

6-1310.3B Intensive vegetated roof systems shall have a minimum growth media depth of 6 inches (150 mm). A maximum growth media depth is not specified for intensive vegetated roof systems. Unless needed to accommodate small trees or large shrubs, the growth media depth should not be greater than 12 inches (300 mm). Intensive vegetated roof systems may include subareas with different growth media depths to accommodate different types of plants.

6-1310.3C The drainage layer below the growth media shall be designed to convey stormwater to the roof downspouts, conductors, and leaders without backing water up into the growth media. Roof areas draining to an individual roof drain may not exceed 4,300 ft^2 (400 m^2) unless internal drainage conduits are provided. Internal drainage conduits shall be designed to convey the 10-year storm.

6-1310.3D Roof drains and emergency overflow measures shall be sized in accordance with the Virginia Uniform Statewide Building Code (VUSBC).

6-1310.3E Vegetated roofs shall have a minimum slope of 2% to provide for adequate drainage. The slope of extensive systems shall not be greater than 33%. The slope of intensive systems shall not be greater than 10%. Extensive systems with slopes equal to or greater than 17% will require supplemental slope stabilization measures (e.g. raised grids) to hold the growth media and plants in place.

6-1310.3F Access to vegetated roofs for maintenance and inspection shall be provided unless waived by the Director. Access shall be provided by an interior stairway through a penthouse or by an alternating tread device with a roof hatch or trap door not less than 16 square feet (1.5 m^2) in area and having a minimum dimension of 24 inches (610 mm), or by a terrace door with a minimum clear opening width of 32 inches (813 mm). The access requirement may be waived by the Director for roofs no greater than 12 feet (3.7 m) above finished grade and less than 1000 square feet (93 m^2) in area.

6-1310.3G Provisions for the safety of maintenance and inspection workers (e.g. parapets, railings, secured rings for safety harnesses, etc.) shall be incorporated in the design of all roofs.

6-1310.3H A vegetation-free zone is recommended along the perimeter of the roof and around all roof penetrations to act as a fire break and to facilitate maintenance and inspection. This zone should be a minimum of 24 inches (61 cm) in width along the perimeter of the roof and a minimum of 12 inches (30.5 cm) around all roof penetrations. The width of the vegetation-free zone around the perimeter of the roof may be reduced from 24 inches (61 cm) to 12 inches (30.5 cm) where application of the 24 inch (61 cm) requirement would result in a reduction of the roof area available for greening of greater than 15%.

6-1310.4 Design and Construction Specifications of Vegetated Roof Components. Vegetated roofs typically consist of the structural components of the roof, a waterproof membrane, a root barrier (if required), a protective layer, a drainage layer, filter fabric, a layer of growth media, and plants. Vegetated roofs may also include an optional thermal insulation layer, a leak detection system, and an irrigation system. Specifications for the optional components of vegetated roofs are not provided herein but should meet any applicable VUSBC requirements. Variations on the vegetated roof system designs in Plates 91-6 and 92-6 (91M-6 & 92M-6) may be approved by the Director provided the facility meets all of the requirements of §6-1310 et seq.

6-1310.4A Waterproof membrane. The waterproof membrane that separates the drainage system and growth media from the structural components of the roof can consist of several different systems including modified bitumen, rubberized asphalt, polyvinyl chloride (PVC), thermoplastic polyolephin (TPO), chlorosulfonated polyethylene (CSPE), and ethylene propylene diene monomer (EPDM) systems. Membranes impregnated with pesticides or herbicides are not allowed. Waterproofing must meet VUSBC requirements.

6-1310.4B Root barrier. A PVC, polypropelene, or polyethelene membrane $\geq$ 30 mil thick hot-air welded at the seams or approved equivalent is required to protect modified bitumen and rubberized asphalt waterproofing from root penetration. A root barrier is not required for PVC, EPDM, or CSPE membranes.

6-1310.4C Protective layer. A perforation resistant protective layer is required to protect the waterproofing and root barrier (if required) from damage. The protective layer shall be a polypropylene non-woven needled fabric with a density (ASTM D3776) $\geq$ 16 oz/yard^2 (542 gm/m^2) and a puncture resistance (ASTM D4833) $\geq$ 220 lbs (979 N) or approved equivalent.

6-1310.4D Drainage layer. The drainage layer shall be a single or composite system capable of conveying stormwater that drains through the growth media. Drainage layers may be a granular drainage media, synthetic geocomposite, or synthetic mat and may include internal drain pipes.

6-1310.4D(1) Granular drainage media shall be a non-carbonate mineral aggregate meeting the requirements listed in Table 6.38.

Table 6.38 Granular Drainage Media Specifications

Saturated Water Permeability (ASTM E2396)	$\geq$ 25 in/min (63.5 cm/min)
Total Organic Matter, by Wet Combustion (MSA)	$\leq$ 1 %
Abrasion Resistance (ASTM C131)	$\leq$ 25 % loss
Soundness (ASTM C88)	$\leq$ 5 % loss
Porosity (ASTM C29)	$\geq$ 25 %
pH	6.5 – 8.0
Grain-size Distribution (ASTM C136) Passing US #8 (2.36 mm) sieve Passing 1/4 in (6.350 mm) sieve Passing 3/8 in (9.525 mm) sieve	 $\leq$ 1% $\leq$ 30% $\geq$ 80%

6-1310.4D(2) For non-grid systems, a drainage system consisting of deformed polyethylene sheet with a transmissivity (ASTM D4716) greater than or equal to 24 gallons per minute per foot (298 liters per minute per meter) of width.

6-1310.4E Filter fabric. Filter fabric shall be a non-woven, root penetrable, needled, polypropylene geotextile meeting the requirements listed in Table 6.39. Heat-set or heat-calendared fabrics are not permitted.

Table 6.39 Filter Fabric Specifications

Unit Weight (ASTM D3776)	$\leq$ 4.25 oz/yd^2 (144 gm/m^2)
Grab Tensile Strength (ASTM D4632)	$\geq$ 90 lbs (400 N)
Mullen Burst Strength (ASTM D3786)	$\geq$ 135 lbs/in^2 (930 kPa)
UV Resistance (ASTM D4355)	70% strength after 500 hours
Permittivity (ASTM D4491)	$\geq$ 2 sec^{-1}

6-1310.4F Growth media. Growth media shall be a mineral and organic mixture that provides sufficient nutrients and water holding capacity to support the proposed plant materials. Tables 6.40, 6.41, and 6.42 provide specifications for the growth media that must be adapted to the specific application by a competent professional.

Table 6.40 Growth Media Specifications

Total Organic Matter by Loss on Ignition (ASTM F1647, Method A)	3 - 15% (dry weight)
Maximum Water Capacity (ASTM E2399)	$\geq$ 45 % (Vol.) for intensive systems $\geq$ 35 % (Vol.) for extensive systems
Non-capillary Pore Space (void ratio) at Field Capacity, 0.333 bar (MSA)	$\geq$ 15% (Vol.)
Saturated Water Permeability (ASTM D2434)	$\geq$ 0.7 in/hr (1.8 cm/hr) for intensive systems $\geq$ 1.4 in/hr (3.6 cm/hr) for extensive systems
pH	6.5 – 8.0

Nitrate + Ammonium, N (in $CaCl_2$)	≤ 80 mg/l
Phosphorus, P_2O_2 (in CAL)	≤ 200 mg/l
Potassium, K_2O (in CAL)	≤ 700 mg/l
Magnesium, Mg (in $CaCl_2$)	≤ 160 mg/l

Table 6.41 Mineral Fraction Grain-size Distribution (ASTM D422) for Intensive Sites

Clay ≤ 0.000079 in (0.002 mm)	3 - 10%
Silt ≤ 0.0029 in (0.075 mm)	10 – 17%
Passing US #60 (0.25 mm) sieve	10 – 40%
Passing US #18 (1.0 mm) sieve	30 – 100%
Passing 1/4 in (6.350 mm) sieve	70 – 100%
Passing 3/8 in (9.525 mm) sieve	90 – 100%

Table 6.42 Mineral Fraction Grain-size Distribution (ASTM D422) for Extensive Sites

Clay ≤ 0.000079 in (0.002 mm)	0%
Silt ≤ 0.0029 in (0.075 mm)	0 – 15%
Passing US #60 (0.25 mm) sieve	10 – 40%
Passing US #18 (1.0 mm) sieve	30 – 100%
Passing 1/4 in (6.350 mm) sieve	70 – 100%
Passing 3/8 in (9.525 mm) sieve	90 – 100%

6-1310.4G Plants. The planting plan and specifications shall be prepared by a certified landscape architect, horticulturist, or other individual who is knowledgeable about the environmental tolerance and ecological functions and impacts of plant species.

6-1310.4G(1) Plant materials selected shall be shallow rooted, self-sustaining, and tolerant of direct sunlight, drought, wind, and frost. Plant materials for extensive systems may include mosses, sedums, herbaceous plants, and grasses. Plant materials for intensive systems may include mosses, sedums, herbaceous plants, grasses, shrubs, and small trees. Invasive species that may disrupt or harm native plant communities shall not be used. The acceptability of proposed plant materials will be determined by the Director. Guidance on the use and selection of plants for vegetated roofs is available from the Urban Forest Management Division.

6-1310.4G(2) Plants may be installed by vegetation mats, plugs, potted plants, sprigs, or direct seeding.

6-1310.4G(3) The planting plan shall be designed to achieve 90 percent coverage within two years of installation.

6-1310.4H Measures for irrigation shall be provided to ensure plant viability during long periods of drought unless waived by the Director. At a minimum, a hose bib shall be provided for manual irrigation. If automated irrigation is provided, the additional dead load shall be incorporated in the roof system design. The requirement to provide measures for irrigation may be waived by the Director for roofs no greater than 12 feet (3.7 m) above finished grade and less than 1000 square feet (93 m^2) in area.

6-1310.5 Construction Requirements.

6-1310.5A The owner shall provide for inspection during construction of the facility by a licensed professional (In accordance with standard practice, the actual inspections may be performed by an individual under responsible charge of the licensed professional). The licensed professional shall certify that the facility was constructed in accordance with the approved plans. The licensed professional's certification along with any material delivery tickets and certifications from the material suppliers shall be submitted to the County prior to bond release. For projects requiring as-built plans, the required certification and supporting documents shall be submitted with or incorporated in the as-built plans. For projects that do not require as-built plans, the required certification and supporting documents shall be submitted with the RUP or non-RUP request.

6-1310.5B Foot and equipment traffic on the roof shall be minimized. Traffic over the waterproof membrane must be strictly controlled until the protective layer and drainage layer are installed.

6-1310.5C The organic and mineral components of the growth media shall be thoroughly mixed prior to installation. It is preferable that the components of the growth media be mixed at a batch facility prior to delivery to the site. The media shall be moistened, as necessary, to prevent separation during installation.

6-1310.5D The growth media shall be soaked at a rate of 30 gallons (114 liters) per 100 square feet (9.3 m^2) and allowed to drain thoroughly before planting.

6-1310.5E Erosion Control. A bio-degradable jute mesh with an aperture of 0.375 - 1.0 inches (9.525 – 25.4 mm) and a tensile strength (ASTM D4632) $\geq$ 20 pounds (89 N) or approved equivalent shall be provided when establishing plants from sprigs and/or seed.

6-1310.5F Plant installation shall occur during the following periods: March 15 through June 15 and September 15 through November 15 unless otherwise approved by the Director.

6-1310.5G Shrubs and potted plants must be hardened off adequately prior to planting.

6-1310.5H The roof should be checked for leakage, slippage of membranes and soil erosion after planting.

6-1310.5I Plantings must be well established prior to release of the conservation deposit. The conservation deposit will be held for a minimum of one year after installation of the plantings and shall only be released if the 90% coverage required by § 6-1310.4G(3) is achieved. If ninety percent coverage is not achieved, the area shall be replanted to achieve the minimum required coverage and the conservation deposit held for an additional year.

6-1310.6 Plan Submission Requirements.

6-1310.6A Plan view(s) showing facility dimensions, planting plan, layout for internal drains (if provided as part of the drainage layer), roof access, walkways, roof penetrations, and setbacks from roof lines.

6-1310.6B Cross section of proposed roof system showing the waterproof membrane, root barrier, protection layer, drainage layer, filter fabric, soil media depth, and emergency overflow system. See Plates 91-6 & 92-6 (91M-6 & 92M-6).

6-1310.6C Specifications for the waterproof membrane, root barrier (if provided), protection layer, drainage layer, filter fabric, and soil media.

6-1310.6D Plant list specifying species, size, and number of proposed plants, seeding rates, planting procedures, and specifications for erosion control.

6-1310.6E Construction requirements, sequence, and procedures including a list of certifications required to be provided to the County.

6-1310.6F Roof area in square feet (m^2) that is vegetated.

6-1310.6G A note shall be placed on the cover sheet stating that the site plan includes a vegetated roof on the proposed building to meet stormwater and water quality control requirements and that construction of the vegetated roof is required with the building. The note shall also state that the building plans shall include a statement signed and sealed by the licensed professional submitting the building design that:

6-1310.6G(1) The vegetated roof design on the building plans is in conformance with the vegetated roof design on the approved site plan;

6-1310.6G(2) Additional requirements for all items such as roof membranes, drains, irrigation systems, and safety rails shall comply with the requirements of the Virginia Uniform Statewide Building Code;

6-1310.6G(3) Access to the vegetated roof has been provided in accordance with Public Facilities Manual § 6-1310.3F;

6-1310.6G(4) Provisions for the safety of maintenance and inspection workers have been incorporated in the design of the vegetated roof in accordance with Public Facilities Manual § 6-1310.3G; and

6-1310.6G(5) Manual or automated irrigation has been provided in accordance with Public Facilities Manual § 6-1310.4H.

6-1311 Reforestation (98-07-PFM)

6-1311.1 (102-08-PFM) Reforestation is the establishment of a forest ecosystem on open ungraded areas. Forest ecosystems reduce the peak rate and volume of stormwater runoff through interception of rainfall by leaves and the forest duff layer, plant uptake and evapotranspiration, and infiltration into the soil. Forest ecosystems improve water quality by capturing and filtering airborne depositional pollutants, plant uptake of dissolved pollutants, and infiltration into the soil. Tree canopies provide energy conservation for buildings, screening, and other benefits in addition to stormwater management. Reforested areas may be used to meet the tree cover requirements of §12-0000 et seq. and Chapter 122 of the Code. Ten-year canopy credit equivalent to the square footage of the area will be given for reforested areas that have been planted, and are established in accordance with the provisions of this section.

6-1311.2 General Requirements.

6-1311.2A Reforested areas shall be placed in restrictive easements that include limited provisions for management practices necessary to assure the establishment of a healthy forest ecosystem. In residential areas, reforested areas must be located on Home Owner Association (or "common") property and may not be located on individual buildable single family detached or attached residential lots, or any part thereof for the purpose of satisfying the detention or water quality control (BMP) requirements of the Subdivision or Zoning Ordinance. Reforested areas may not be located on individual residential lots in non-bonded subdivisions to satisfy the BMP requirements of the Chesapeake Bay Preservation Ordinance.

6-1311.2B Reforested areas shall be privately managed and maintained.

6-1311.2C Post-development hydrology. A runoff coefficient "C" for reforested areas found in Table 6.6 shall be used for hydrologic computations using the Rational Method. The Curve Number "CN" for use with Natural Resources Conservation Service (NRCS) methods shall be based upon woods in good condition and the underlying Hydrologic Soil Group.

6-1311.2D Reforested areas shall be posted with permanent signs designating the area as a Conservation Area. Signs shall state that the area has been reforested as a Low Impact Development practice and no disturbance or cutting of vegetation is allowed. Signs must be a minimum of 8 inches (205 mm) by 10 inches (254 mm) mounted on posts at a height of four (4) feet (1.22 m) to six (6) feet (1.83 m) and placed at approximately 150 foot (46 m) intervals along the perimeter of the reforested area. See Plates 81-6 and 81M-6.

6-1311.2E In order to maximize the infiltration capacity, structure, and biota of the existing soil profile below the amended soil layer, areas to be reforested shall not be graded as part of the site development. The only land disturbance allowed is that which is necessary to amend the soils and install plantings.

6-1311.3 Design of Reforested Areas.

6-1311.3A Reforestation plans and specifications shall be prepared by a certified landscape architect, horticulturist, or other individual who is knowledgeable about the environmental tolerance, ecological functions, and impacts of plant species.

6-1311.3B Except as noted below, reforested areas shall have a minimum contiguous area of 6,000 square feet (557 m^2), be generally regular in shape, and have a minimum width of 35 feet (10.7 m). The Director may approve areas less than 6,000 square feet (557 m^2) in size or with minimum widths less than 35 feet (10.7 m) provided such areas are contiguous to existing naturally vegetated areas that are preserved with restrictive easements or other long-term protective mechanisms or that are in uses associated with long-term preservation.

6-1311.3C Reforested areas shall be designed to replicate adjacent forest communities using similar percentages of major indicator species or species that can adapt to abiotic conditions present in the area to be reforested. If there is no adjacent forest community to mimic, the area may be planted with pioneer species, such as Virginia pine, black locust, eastern redcedar, red maple, and persimmon.

6-1311.3D Reforested areas shall consist of a mixture of overstory trees, understory trees, and shrubs. Generally, overstory trees correspond to Category 3 or 4 trees and understory trees correspond to Category 1 or 2 trees as listed in Table 12.7 in §12-0000 et seq. At least 25% of the area shall be planted with trees from nursery stock. For nursery stock, deciduous trees must be a minimum of 1" (2.54 cm) caliper and evergreen trees must be a minimum of 6 feet (1.8 m) in height. For areas planted with nursery stock, the density of overstory trees shall be a minimum of 100 trees per acre and the density of understory trees shall be a minimum of 200 trees per acre. Nursery stock may be replaced by transplanted material as approved by the Director. For areas planted with bareroot seedlings (See § 12-0805.5A), the density of the trees shall be double that required for nursery stock. The density of shrubs shall be a minimum of 1089 plants per acre. Shrubs must be a minimum of 18 inches (0.4 m) in height.

6-1311.3E To curtail the spread of disease or insect infestation in a plant species, no more than 70% of the trees, seedlings, and shrubs required to be planted shall be of one genus. In addition, no more than 35% of the deciduous trees shall be of a single species and no more than 35% of the evergreen trees shall be of a single species. Seedlings shall be randomly mixed and placed approximately 8 - 10 feet (2.4 - 3.1 m) apart in a random pattern with shrubs placed surrounding seedlings. Additional guidance on appropriate species for soils and groundwater conditions can be found in § 12-0000.

6-1311.3F Tree planting credit may be given for existing trees within the planting area. A planting credit of one (1) 1 inch (2.54 cm) caliper nursery stock overstory tree shall be given for each 150 square feet (14 m^2) of existing overstory tree canopy and a planting credit of one (1) 1 inch (2.54 cm) caliper nursery stock understory tree shall be given for each 75 square feet (7 m^2) of existing understory tree canopy.

6-1311.3G Compacted soils will limit root growth and establishment of the forest ecosystem. Subsoiling (tilling) and soil amendments are required to relieve soil compaction and restore soil function in previously disturbed soils except as noted below.

6-1311.3G(1) Subsoiling and soil amendments are not required if the in situ bulk density of the existing soil, as measured by the sand cone test (ASTM D1556), is less than the value in Table 6.43 for the corresponding soil type or compaction, as measured by the cone penetration test (ASTM D3441), is less than 300 lb/in^2 (2068 kPa) in the top 15 inches (38 cm) of soil. A minimum of one density measurement or test shall be performed per 1,000 ft^2 (93 m^2).

6-1311.3G(2) Testing of in situ soils to determine compaction is not required if soils will be amended at pre-approved rates in accordance with § 6-1311.5.

Table 6.43
Bulk Densities That May Effect Root Growth[3]

Soil Texture	(lb/ft^3)	(gm/cm^3)
Sands, loamy sands	105.50	1.69
Sandy loams, loams	101.76	1.63
Sandy clay loams, loams, clay loams	99.88	1.60
Silts, silt loams	99.88	1.60
Silt loams, silty clay loams	96.76	1.55
Sandy clays, silty clays, some clay loams (35-45% clay)	93.02	1.49
Clays (>45% clay)	86.77	1.39

[3] From *Protecting Urban Soil Quality: Examples for Landscape Codes and Specifications* USDA 2003

6-1311.4 Subsoiling and Soil Amendment Specifications.

6-1311.4A The topsoil layer shall have a minimum depth of 8 inches (20 cm) except for areas within the dripline of existing trees in or adjacent to the area to be reforested, where subsoiling may adversely impact tree roots. Compacted soils within the dripline of existing trees shall be addressed by the use of vertical mulching as specified in § 12-0806.4B. See Plates 1-12 & 13-12 (1M-12 & 13M-12).

6-1311.4B Subsoils below the amended topsoil layer shall be scarified to a depth of at least 4 inches (10 cm).

6-1311.4C A minimum of 2 inches (5 cm) of organic mulch shall be placed on the topsoil layer at final grade for stabilization purposes after planting. Mulch shall consist of wood chips, bark chips, or shredded bark, that has been aged for a minimum of 4 months. Alternatively, a native seed mixture combined with appropriate stabilization measures may be used.

6-1311.5 Construction Specifications for Subsoiling with Soil Amendments. The following construction specifications are designed to achieve an 8 inch (20 cm) depth of topsoil and scarification of compacted subsoil 4 inches (10 cm) below the topsoil layer for a total uncompacted depth of 12 inches (30 cm).

6-1311.5A Scarify or till subgrade to 8 inches (20 cm) depth. The entire surface shall be scarified except for the area within the drip lines of existing trees to be retained.

6-1311.5B Place and rototill 3 inches (8 cm) of organic compost meeting the requirements of Table 6.32 into 5 inches (13 cm) of soil.

6-1311.5C Water thoroughly and allow soil to settle for one week.

6-1311.5D Rake beds to smooth and remove surface rocks larger than 2 inches (5 cm) in diameter.

6-1311.5E Planting should occur as soon as feasible after the soil has been amended.

6-1311.5F After planting, mulch planting beds with 2 inches (5 cm) of organic mulch. Mulch shall consist of wood chips, bark chips, or shredded bark, that has been aged for a minimum of 4 months. Alternatively, a native seed mixture combined with appropriate stabilization measures may be used. Installation of the above stabilization measures shall be in accordance with the Virginia Erosion and Sediment Control Handbook, 3rd edition, 1992.

6-1311.5G An inspection report shall be provided to DPWES for review prior to bond release. This report shall include observed survival rates of plantings, replacement plantings installed, material delivery tickets, and certifications from material suppliers. For projects requiring as-built plans, the required inspection report and supporting documents shall be submitted with or incorporated in the as-built plans. For projects that do not require as-built plans, the required report and supporting documents shall be submitted with the RUP or non-RUP request.

6-1311.6 Planting Requirements.

6-1311.6A Planting procedures for trees, shrubs and seedlings shall be in conformance with § 12-0805.

6-1311.6B Planting of the reforested area should be done with minimal mechanical disturbance to the existing trees and shrubs to be preserved and given credit per § 6-1311.3F. The planting should be done by hand or mechanical auger.

6-1311.6C Plantings must be well established prior to release of the conservation deposit. The conservation deposit will be held for a minimum of two years after the initial installation of the plantings. Ninety percent or more of the minimum number of nursery stock trees and shrubs required by the approved plan shall be viable (healthy and capable of developing a trunk and branch structure typical for their species) at the time the conservation deposit is released. Sixty-seven percent or more of the initial tree seedling density required by the approved plan shall be viable at the time the conservation deposit is released. If these minimum percentages are not met at the time of inspection, additional nursery stock trees, nursery stock shrubs, and seedlings shall be planted at densities necessary to achieve the required minimum percentages of viability of the initial plantings based on the observed mortality rates. For example, if the plan called for 500 seedlings to be planted, a minimum of 335 seedlings (67%) must be viable more than 2 years after installation. If 250 seedlings were viable (a deficit of 85 viable plants) at the time of inspection (2.5 years after installation), 170 replacement seedlings would need to be planted, based on the observed mortality rate (50%), prior to release of the conservation deposit. Replacement seedlings shall be selected such that the resulting mixture of surviving and replacement plants will generally achieve the mixture of understory trees and overstory trees shown on the approved plan as determined by the Director.

6-1311.7 Plan Submission Requirements.

6-1311.7A Plant schedule and planting plan specifying species, quantity of each species, stock size, type of root stock to be installed, and spacing of proposed plants within the reforested area.

6-1311.7B Reforested areas shall be delineated on the plan sheets with the note: "Reforestation Area. This area is being replanted for Low Impact Development credit. No disturbance other than that necessary to implement the planting plan allowed."

6-1311.7C Construction specifications for soil amendments (if provided) and planting procedures.

6-1311.7D In situ soil test results if performed (See § 6-1311.3G).

6-1400 FLOODPLAIN

6-1401 Requirements

6-1401.1 In areas of streambeds subject to inundation with 70 acres (28.33 ha) or more of watershed, floodplain water surface elevations shall be computed in order that floodplain easements may be established.

6-1401.2 Where open drainage swales exist and drainage improvements are not provided, a drainage study and storm drainage easement to cover the 100-

yr drainageway must be provided on watersheds less than 70 acres (28.33 ha).

6-1402 Flows

6-1402.1 All floodplains shall be calculated for a quantity of runoff based on the 100-yr design storm.

6-1402.2 Flows in floodplains shall be determined by the methods discussed in § 6-0800 et seq.

6-1403 Methods and Guidelines for Calculations (Plate 45-6 (45M-6))

6-1403.1 Water surface elevations may be determined by the Standard Step Method which is expressed as:

$$\mathrm{B} = \left(\mathrm{Z}_2 + \mathrm{d}_2 + \mathrm{V}_2{}^2/2\mathrm{g}\right) - \left(Z_1 + d_1 + V_1{}^2/2g - H_L\right)$$
$$\left(\mathrm{B} = \left(\mathrm{Z}_2 + \mathrm{d}_2 + \mathrm{V}_2{}^2/2\mathrm{g}\right) - \left(Z_1 + d_1 + V_1{}^2/2g - H_L\right)\right)$$

$$V = \frac{Q}{A} = \frac{1.49}{n} r^{2/3} S^{1/2}$$
$$\left(V = \frac{Q}{A} = \frac{r^{2/3}}{n} S^{1/2}\right)$$

$$H_L = 1/2\left(S_1 + S_2\right)L = 1/2\left\| \frac{(n_1 v_1)^2}{2.208 r_1{}^{4/3}} + \frac{(n_2 v_2)^2}{2.208 r_2{}^{4/3}} \right\| L$$
$$\left(H_l = 1/2\left(S_1 + S_2\right)L = 1/2\left\| \frac{(n_1 v_1)^2}{r_1{}^{4/3}} + \frac{(n_2 v_2)^2}{r_2{}^{4/3}} \right\| L\right)$$

6-1403.2 With exception of the term B, above terms are defined on Plate 45-6 (45M-6).

6-1403.3 The term B is the balance of energy between the 2 sections which shall be 0.2' (0.06m) + or -.

6-1403.4 The method is a trial and error procedure throughout most of the floodplain. General guidelines to performing calculations are as follows:

6-1403.4A Select floodplain cross-sections.

6-1403.4A(1) (61-98-PFM) These sections shall be selected based on the topography and any existing and/or proposed hydraulic control sections. Floodplain cross-sections shall be developed from field run data or aerial spot elevations in accordance with methods described in the *Flood Insurance Study Guidelines and Specifications for Study Contractors* (Federal Emergency Management Agency, January 1995). Elevation data for all hydraulic structures and buildings in or adjoining the floodplain shall be collected by field survey. Areas of the channel which are underwater shall be field surveyed unless permission is granted in advance by the Director to use the water surface as the assumed channel bottom. Upon written request, permission to use the water surface as the assumed channel bottom may be granted by the Director where the assumption will not affect the Director's ability to administer the floodplain regulations. Topography between the cross-sections shall be at 2' (0.5m) contour intervals and may be developed by field or aerial methods. Aerial survey methods shall meet the *American Society of Photogrammetry and Remote Sensing Accuracy Standards for Large-Scale Maps* (ASPRS, 1990).

6-1403.4A(2) Cross-sections are needed at floodplain contractions, expansions, sharp changes in invert slope, and where abrupt changes in channel roughness occurs.

6-1403.4A(3) Special care shall be taken to include the effects of all major constrictions (such as culvert crossings under roads, etc.) in the computations.

6-1403.4A(4) Distance along the baseline between sections shall not exceed 300' (90m). Locations of cross-sections is subject to approval of the Director, therefore, cross-sections selected shall be coordinated.

6-1403.4B Cross-sections shall be as nearly perpendicular to floodplain flow as possible. The baseline shall be located as close as possible to the center of the flood area.

6-1403.4C The roughness coefficients, (n), for the floodplain shall be obtained from DPWES. The designer may be permitted to use different values of roughness coefficients for the center of the stream and the overflow banks of each cross-section.

6-1403.4D If the floodplain study is being prepared for a particular site or property, the floodplain shall extend downstream a minimum of 300' (90m) from the lower property line or to a control section. The floodplain shall extend a minimum of 300' (90m) upstream from the upper property line.

6-1403.4E When the floodplain study is prepared in accordance with the provisions of Parts 6 and 9 of Article 2 of the ZO, then, consideration of the effects of any proposed use shall be based on the assumption that there will be an equal degree of encroachment by others extending for a significant reach on both sides of the stream. This combined effect must not have an adverse effect (normally construed to include no rise in water surface elevation) upon the adopted 100-yr floodplain.

6-1404 Water Surface Calculations

6-1404.1 Water surface calculations shall begin where the energy and hydraulic gradient are known or can readily be obtained.

6-1404.2 Calculations shall generally be performed in an upstream direction since floodplains are usually subcritical flow through the entire floodplain.

6-1404.3 Once the water surface is established at a cross-section, the water surface in the next cross-section is assumed, the total energy (distance to the EGL) is calculated, and then the energy balance between the 2 cross-sections is computed.

6-1404.4 If the energy balance does not meet the required accuracy, then assume another water surface elevation and repeat calculations.

6-1404.5 When the energy balance meets the required accuracy, the water surface elevation is established and calculations may proceed between the next 2 cross-sections.

6-1405 Floodplain Easement

6-1405.1 All floodplains, or portions of floodplains, that pass through a project site shall have a floodplain easement. This easement shall be shown on the plats and plans and shall be designated as a "Floodplain and Storm Drainage Easement." The following note also must be clearly shown: "No use shall be made of, nor shall any improvements be made in, the floodplain easement without specific authorization from Fairfax County."

6-1405.2 The easement shall be placed around the water limits as established by the floodplain calculations. This easement shall be tied to the site boundaries in such a manner that the easement may be established at the site.

6-1405.3 The floodplain easements shall be placed on the record plat, the site construction drawings and floodplain study. However, only the record plat is required to have the metes and bounds of the easements and the boundary tie information.

6-1500 ON-SITE MAJOR STORM DRAINAGE SYSTEM

6-1501 Guidelines for Major Drainage System. Major and minor storm drainage systems are defined in § 6-0100 et seq.

6-1501.1 For most developments, the on-site major storm drainage system is the natural backup system and therefore consists of the less obvious drainageways.

6-1501.1A It is desirable that the major system provide drainage relief such that no buildings will be flooded with a 100-yr design flow, even if the minor system should fail due to blocking.

6-1501.1B If a minor drainage system is available, the major system shall be designed to carry the runoff in excess of the capacity of the minor system.

6-1501.2 Guidelines for design of the on-site major drainage system are as follows:

6-1501.2A Areas shall be graded in such a manner and/or buildings located or constructed in such a manner that if a complete failure of storm sewer system occurs, no building will be flooded by the design flow.

6-1501.2B Key areas to watch are, sump areas, relatively flat areas, and areas where buildings are located below streets and/or parking lots.

6-1501.2C The 100-yr frequency storm shall be used to compute the runoff for the major drainage system.

6-1501.2D For the first trial, the same time of concentration values that were used in designing the minor drainage system shall be used and the minor system shall be assumed to be completely inoperable. If no building will be flooded based on these assumptions, then the analysis may be considered complete.

6-1501.2E If buildings will be flooded based on the assumptions used in § 6-1501.2D then the designer should perform more precise hydrologic and hydraulic computations. He shall design the minor system, overland relief swales, and/or surface storage in such a way that no building will be damaged by flooding.

6-1501.2F The minor storm drainage system normally should not be oversized as a design for the major system. The major drainage system should be in the form of grading of the area and/or locating and constructing buildings in such a manner that overland relief swales and/or surface storage will accomplish the objective. In some instances where a sump condition exists, the design engineer may desire to locate storm sewer openings and structures below the overland relief elevation.

6-1502 Major Drainage System Design Calculation

6-1502.1 In general the design guidelines described in § 6-1500 et seq. are intended to result in a functional analysis rather than a numerical analysis.

6-1502.2 Design engineers shall indicate on the project drainage plans, the location and approximate extent of the overland relief path and areas that may be affected by surface storage for a 100-yr design storm. Overlaying arrows, shading or other suitable see-through graphics are suggested for this purpose.

6-1502.3 Any calculations that may have been necessary in order to arrive at the major system shall be submitted.

6-1503 Overlot Grading in Residential Areas

6-1503.1 Grading plans for construction of dwellings shall show proposed grading necessary to ascertain adequate drainage and to show that overland relief will be provided.

6-1503.2 Compliance with the grading plan shall be required to the extent that zoning yard requirements are maintained, adequate drainage is provided and the limit of clearing is honored.

6-1503.3 (27-89-PFM) In residential areas, on lots of 36,000 ft² (3344 m²) or less, where a developer creates cut or fill slopes 3:1 or steeper, with unbroken vertical heights of 4' (1.2m) or greater, such slopes shall be stabilized with a ground cover selected by the developer from the following list:

English Ivy, Ajuga, Crown Vetch, Pachysandra, Periwinkle, Euonymus, Creeping Juniper or other as approved by the Director.

Since ground covers normally require 2 yr for establishment, the following apply:

6-1503.3A Interim ground protection that will not inhibit development of the ground cover must be provided and be effective before RUP is issued. This protection may be a temporary grass cover or a permanent mulch providing that it does not interfere with growth of the ground cover.

6-1503.3B Because of the establishment time, the conservation deposit approved by the County posted prior to plan approval (subdivision or individual lot grading plan) shall include an amount equal to the estimated cost of seeding or sprigging with the selected ground cover all areas which qualify for such cover. The deposit shall not be released until all slopes scheduled for ground cover are covered to the satisfaction of the Director.

6-1503.3C If the first purchaser of a residential lot of 36,000 ft² (3344 m²) or less requests, in writing, that the above types of ground cover not be provided on the lot, and accepts the maintenance responsibility therefore, the Director may authorize this exception.

6-1503.3D The alternate selected by the buyer, either grass or permanent hardwood bark mulch, must be completely established/installed before a RUP can be issued.

6-1503.4 On residential lots exceeding 36,000 ft² (3344 m²), the ground stabilization to be provided on all disturbed areas regardless of slope shall normally be a permanent variety of grass unless otherwise agreed to in writing between the buyer and the developer.

6-1503.5 For all industrial and commercial areas, the ground stabilization to be provided for all disturbed areas regardless of degree of slope shall normally be a permanent grass; except that mulches and ground covers may be accepted with specific approval of the Director.

6-1503.6 For all slopes in VDOT rights-of-way, regardless of zoning, the ground stabilization shall be a permanent grass acceptable to VDOT unless otherwise specifically approved by VDOT.

6-1503.7 (27-89-PFM) Regardless of location, no mulch or vegetation stabilized slopes steeper than 2H:1V shall be approved for vertical heights greater than 4' (1.2m) without intervening retaining walls or 4' (1.2m) wide benches. The Director may specify which option shall be provided, or may require a combination of the two. The Director's written authorization under this section must be obtained.

6-1600 DESIGN AND CONSTRUCTION OF DAMS AND IMPOUNDMENTS (46-94-PFM)

6-1601 Virginia Dam Safety Regulations

6-1601.1 (84-04-PFM) Construction of impoundments with a dam height of 25' (7.6m) or greater and with an impoundment capacity of 15 acre-feet (18,502 m^3) or more, and impoundments with a dam height of 6' (1.8 m) or greater and with an impoundment capacity of 50 acre-feet (61,672 m^3) or more, requires compliance with the Virginia requirements set forth in the Virginia Soil and Water Conservation Board's Impounding Structure Regulations (VR 625-01-00), dated February 1, 1989, as revised under 4VAC 50-20-30, effective July 1, 2002. Definitions of "dam height" and "impoundment capacity" per the Virginia Dam Safety Regulations are as follows: "Dam Height" means the structural height of an impounding structure, and is the vertical distance from the natural stream bed or watercourse measured at the downstream toe of the impounding structure to the top of the impounding structure; "Impoundment Capacity" means the volume in acre-feet (cubic meters) that is capable of being impounded at the top of the impounding structure.

6-1601.2 Permits for construction and operation of these dams are issued by the Virginia Soil and Water Conservation Board.

6-1601.3 A copy of any state-approved design also must be submitted to the Director in order to receive Director approval for grading and/or construction plans. Grading or construction plans must also include an Erosion and Sediment Control (E&S) plan in accordance with the provisions in § 11-0000 (Erosion and Sediment Control) et seq.

6-1602 County Dam Safety Regulations

6-1602.1 The Director shall require compliance with the procedures and criteria set forth in § 6-1602 through § 6-1608 for the design and construction of new dams and the rehabilitation of existing dams in the County that are not under the jurisdiction of a Federal agency or Virginia. The rehabilitation of a feature of a dam or related facilities which would require construction or grading plan approval shall conform to the criteria and procedures which apply to that particular feature. However, where the rehabilitation of the feature would bring into question the safety or functional capability of another feature(s), as determined by the Director, that feature(s) shall also be rehabilitated to comply with the criteria and procedures. Rehabilitation means restoring the dam and/or appurtenant structures to the original or a superior functional condition affecting the design, construction, operation and/or performance, in a manner requiring the prior submission to the County of a construction or grading plan.

6-1602.2 These guidelines and standards are intended to ensure public safety and welfare. Dams are complex structures which must be designed and constructed taking into account specific site conditions, the characteristics of the construction materials, the particular functions of the dam and the hazards associated with the particular site. No written document can cover all design and construction problems that may be confronted by the design engineer. The acceptability of the design and the adequacy of the plans and specifications will be made on a case-by-case basis. The primary responsibility for the proper design of the dam and appurtenant structures shall continue to be that of the design engineer.

6-1602.3 The County will regulate all dams except existing or proposed dams owned, operated, and maintained by the federal government or Virginia.

6-1602.4 All dams formed by highway embankments are also subject to the following additional procedures and criteria:

6-1602.4A VDOT special design considerations for permanently impounding water upstream of highway embankments.

6-1602.4B VDOT approval and acceptance must be secured for these impoundments.

6-1602.5 (88-05-PFM) VDOT will consider accepting subdivision streets for maintenance that occupy dams when the dam criteria outlined in the VDOT Subdivision Street Requirements are met. A roadway will be considered to occupy a dam if any part of the fill for the roadway and the fill for the dam overlap or if the area between the two embankments is filled in so that the downstream face of the dam is obscured or if a closed drainage facility from a dam extends under a roadway fill.

6-1602.6 (88-05-PFM) For privately maintained dams, a Private Maintenance Agreement shall be executed with the County by the owner of the dam and recorded among the land records of the County prior to construction plan approval.

6-1602.7 (88-05-PFM) A storm drainage easement shall be provided sufficient to convey the maximum emergency spillway flow downstream to an adequate drainage system (see § 6-0200). The intent of this provision is to restrict future development within the area immediately downstream from the spillway that will be inundated based on the appropriate spillway design flood.

6-1602.8 (88-05-PFM) Dams regulated by the County shall be designed by a Professional Engineer licensed in Virginia with experience and expertise in the fields of hydrology, hydraulics and geotechnical engineering.

6-1602.9 (88-05-PFM) The owner of the dam shall comply with the requirements of § 6-1607 related to the construction, inspection, and as-built certification of dams regulated by the County.

6-1603 Hydrologic Design Criteria for Dams Regulated by the County

6-1603.1 The emergency spillway shall be designed to safely pass or store the spillway design flood (SDF) without overtopping the dam. In addition, a freeboard shall be established in accordance with the criteria set forth below:

6-1603.1A The SDF shall be determined based on a spillway design storm determined from Plates 46-6 (46M-6) and 47-6 (47M-6). The spillway design storm total rainfall amount shall also be determined from Plate 46-6 (46M-6). The minimum storm duration shall be 24-hr. A storm hyetograph shall be constructed using the Soil Conservation Service (SCS) 24-hr duration, Type II rainfall distribution shown in Plates 47-6 (47M-6) and 48-6 (48M-6). Once the spillway design storm hyetograph is constructed, the SDF hydrograph shall be determined using standard SCS unit hydrograph techniques.

6-1603.1B (57-96-PFM) A freeboard above the water-surface elevation resulting from the SDF shall be determined based on a freeboard hydrograph (FBH) developed using the next highest storm (i.e., total rainfall amount) from Plate 46-6 (46M-6). The storm duration and storm distribution for the FBH shall be the same as that used for the SDF. The top of dam elevation should be designed at or above the water-surface elevation resulting from routing the FBH. For Class A, B and C reservoirs (see Plate 64-6 (64M-6)), the minimum freeboard shall be no less than 2' (0.6m) above the SDF elevation. For Class D reservoirs with drainage areas less than 20 acres (8 ha), the minimum freeboard shall be no less than 1' (0.3m) above the SDF elevation; however, the Director may require more freeboard if there is potential for downstream property damage or personal injury.

6-1603.1C The SDF and FBH shall be routed through the impoundment assuming that no storage is available below the emergency spillway crest and that the principal spillway is inoperative or clogged.

6-1603.2 An emergency spillway separate from the principal spillway should be provided. The Director may allow a combined principal/emergency spillway if existing conditions (such as the necessity to cut through rock) dictate. When a vegetated overland emergency spillway is proposed, the frequency of use for the emergency spillway shall be limited based on the following hydrologic criteria:

6-1603.2A For dry impoundments having drainage areas less than 70 acres (28 ha), dam heights less than or equal to 15' (4.5m) and impoundment capacities less than or equal to 25 acre-feet (30850 m^3), the principal spillway shall convey the entire 10-yr flood and the emergency spillway crest shall be set at or above the 10-yr flood elevation.

6-1603.2B For all other dry impoundments, the emergency spillway crest shall be set at or above the 25-yr flood elevation and the principal spillway shall convey the entire 25-yr flood.

6-1603.2C For wet impoundments having drainage areas less than 70 acres (28 ha), dam heights less than or equal to 15' (4.5m) and impoundment capacities less than or equal to 25 acre-feet (30850 m^3), the emergency spillway crest shall be set at or above the 25-yr flood elevation and the principal spillway shall convey the entire 25-yr flood.

6-1603.2D For all other wet impoundments, the emergency spillway crest shall be set at or above the 50-yr flood elevation and the principal spillway shall convey the entire 50-yr flood.

6-1603.2E The 10-, 25-, and 50-yr recurrence interval floods mentioned in § 6-1603.2A thru § 6-1603.2D shall be developed as hydrographs using a minimum 24-hr storm duration, rainfall amounts from Table 6.23, storm distribution from Plate 47-6 (47M-6), and standard SCS unit hydrograph techniques for converting the rainfall hyetograph to a runoff hydrograph.

6-1603.3 When 2 or more dams are positioned in a series, the following criteria shall apply:

6-1603.3A Upper dam. The hydrologic design criteria for the design of the upper dam in a system of dams in series shall be the same as, or more stringent than, those for the dams downstream, if failure of the upper dam could contribute to failure of the lower dam.

6-1603.3B Lower dam(s). For the design of a lower dam in a system of dams in a series, hydrographs shall be developed for the areas controlled by the upper dams based on the same hydrologic criteria as the lower dam(s). The hydrographs shall be routed through the spillways of the upstream dams and the outflows routed to the lower dam where they are combined with the hydrograph from the intermediate uncontrolled drainage area. The combined emergency spillway hydrograph and the combined freeboard hydrograph shall be used to determine the size of the emergency spillway and the height of dam at the lower site. If upon routing a hydrograph through the upper dam, the dam is overtopped or its safety is a concern, as determined by the Director, it shall be considered breached. For design of the lower dam, the breach hydrograph shall be routed downstream to the lower dam and combined with the uncontrolled area hydrograph. The breach hydrograph shall be determined as described in § 6-1603.4B through § 6-1603.4D.

6-1603.4 As a part of the overall dam design, the engineer shall determine the segment of the stream valley downstream from the dam that would experience an increased flood depth resulting from a potential dam failure. This analysis may be waived by the Director for those dams having drainage areas less than 70 acres (28 ha), dam heights less than 15' (4.5m) and impoundment capacities less than 25 acre-feet (30850 m^3). However, the Director can require that a dam breach analysis be performed for any size impoundment if there is any concern as to the potential hazard that the impoundment presents to downstream properties. The following procedure shall be used to perform the dam breach analysis:

6-1603.4A Initially, a dam breach analysis assuming failure due to internal erosion (piping) shall be performed. The reservoir level will be assumed to be at the crest of the emergency spillway for this analysis. This type of analysis is sometimes referred to as a "sunny day" breach since pond inflow is assumed to be equal to 0. The resulting dam breach hydrograph shall be routed downstream of the dam to a point where the dam break flood depth has attenuated to a depth less than the 100-yr flood elevation.

6-1603.4B After performing the "sunny day" dam breach analysis, a dam overtopping breach analysis shall be performed. The storm amount from the chart in Plate 46-6 (46M-6) which overtops the proposed dam shall be the basis of analysis. After determining the overtopping storm from the chart in Plate 46-6 (46M-6), a dam breach hydrograph shall be developed assuming that the dam fails at the time of maximum water-surface elevation in the reservoir. This dam breach hydrograph shall be routed downstream to a point where the dam break flood depth has attenuated to within 1' (0.30m) or less than the flood depth that would be experienced without the dam.

6-1603.4C Methods used by the Corps of Engineers, the Soil Conservation Service, or the National Weather Service for computing the outflow hydrograph resulting from a dam failure, or other methods approved by the Director, may be used for this analysis.

6-1603.4D The following guidelines are offered for performing dam break analysis for earth dams when failure results from overtopping and when using methods that require assumptions regarding the dam breach shape and time to failure. Analysis using the

HEC-1 computer program is recommended when simulating a failure due to overtopping.

6-1603.4D(1) Breach Width (the width at the bottom of the breach when breach is at maximum size): The ultimate breach width for an earth dam can vary greatly; however, the breach width should be between 1/2 the dam height and 4 times the dam height. A breach width ranging from 2 to 3 times the dam height may be used in most situations. The breach width can be a function of reservoir volume. For instance, for 2 dams having the same height and section but exceedingly different impoundment capacities, the dam having the smaller capacity would normally have the smaller breach width. The ultimate breach width will also be limited by the size of the natural stream valley where the dam is located.

6-1603.4D(2) Side Slope of Breach (Z=Horizontal to One Vertical):

$$0 < Z < 1.$$

6-1603.4D(3) Failure Time (T_F). Failure time will vary dependent on dam section, embankment material, and impoundment size. For typical dam embankments with minimum required top widths, a failure time based on an erosion rate of 2 ft (0.6m)/minute may be used.

6-1603.4D(4) Pool Elevation at Which Failure Begins ($FAIL_{ELEV}$): Failure typically begins at a depth of 1' (0.30m) to 5' (1.50m) above the dam top. However, for the purposes of the dam break analysis, failure shall be assumed to begin at the maximum pool elevation achieved during the overtopping flood.

6-1603.4E If the dam break analysis shows a potential for flooding of structures, the engineer shall increase the spillway capacity in excess of the requirements shown in Plate 46-6 (46M-6), at least to the point where there is no potential flooding of structures, or reduce the dam height to a level which results in no increase in flooding of structures.

6-1603.4F If the dam break analysis shows no increase in flood depths, the spillway capacity may be reduced to a level where flood depths start to increase. In such a case, the engineer must provide computations and a narrative on the plans supporting the reduced SDF and FBH criteria. The minimum acceptable SDF shall be the 100-yr flood. Any reduction of the SDF and FBH criteria will require the explicit approval of the Director.

6-1604 Design Guidelines for Spillways

6-1604.1 Drop Inlet Spillways. Drop inlet spillways generally consist of a riser structure located in the reservoir area connected to a pipe or box culvert (outlet conduit) which extends through the dam embankment. Drop inlet spillways should be designed such that full flow is established in the outlet conduit and riser at as low a head over the riser crest as practical (see § 6-1604.1B), and to operate without excessive surging, noise, vibration , or vortex action at any stage. This requires that the riser have a larger cross-sectional area than the outlet conduit. The following procedures should be used for computation of the discharge rating curve for a drop inlet spillway.

6-1604.1A The following general equations are for flow determination at various locations throughout a drop inlet spillway.

6-1604.1A(1) Weir flow (rectangular cross section):

$Q = CLH_w^{3/2}$
($Q = 0.552 \times CLH_w^{3/2}$)
Q = discharge, CFS (CMS)
C = weir flow coefficient, typically set at 3.0 but may vary with head and weir shape
L = weir length, ft (m)
H_w = energy head above spillway crest, ft (m)

This equation is applicable for the initial stages of flow over the riser crest as well as initial stages of flow into rectangular ports along the riser column.

6-1604.1A(2) Orifice flow:

$Q = CA(2gH_o)^{1/2}$
($Q = CA(2gH_o)^{1/2}$)
Q = discharge, CFS (CMS)
C = orifice coefficient, typically set at 0.6 for sharp edged orifices but may vary depending on orifice geometry
A = flow area, ft² (m²)
g = acceleration of gravity, 32.2 ft/sec.² (9.81 m/sec.²)
H_o = energy head above centroid of opening, ft (m)

This equation is applicable for flow through openings on the spillway riser which are totally submerged and no longer operating under weir flow.

6-1604.1A(3) (57-96-PFM) Outlet conduit pipe flow. The outlet conduit should be designed to flow full with control occurring at the outlet. In most cases, the following equation can be used to determine the energy loss through a principal spillway conduit operating under outlet control:

$$H = \left[1 + k_e + k_b + \frac{29n^2L}{R^{4/3}}\right]\frac{V^2}{2g}$$

$$\left(H = \left[1 + k_e + k_b + \frac{0.05n^2L}{R^{4/3}}\right]\frac{V^2}{2g}\right)$$

H = head loss, ft (m)
k_e+k_b = entrance and bend loss coefficients, typically set at 0.7, but may vary depending on entrance and bend geometry
n = Manning's roughness coefficient, typically set at 0.013 for concrete
L = conduit length, ft (m)
R = hydraulic radius of conduit, ft (m)
V = flow velocity in conduit, fps (mps)
g = acceleration of gravity, 32.2 ft/sec.^{2} (9.81 m/sec.^{2})

In some cases, particularly if the outlet conduit is set at a steep slope, full flow will not occur in the pipe conduit and control may occur at the junction between the outlet conduit and the riser. Calculation of a rating curve with the control at this location can be estimated by assuming orifice flow into the outlet conduit and using the orifice equation or by using FHA inlet control nomographs. It should be understood that the inlet control nomographs are not truly representative of this type of flow situation and should be used with the understanding that they were developed to predict flow through highway culverts operating under inlet control. However, depending on the size relationship between the riser and outlet conduit, the inlet control nomograph may provide a reasonable estimate.

6-1604.1B The drop inlet spillway shall be designed so that full flow is established prior to the occurrence of orifice flow at the riser top. Therefore, for any Q, the water surface elevation associated with orifice flow at the riser top must be less than the water surface elevation resulting from either weir flow at the riser rim or full flow in the outlet conduit. A design that results in full flow occurring at as low a head over the riser top as practical is superior. In most cases, drop inlet spillways should be designed such that full flow is established in both the outlet conduit and riser. This type of design will discourage excessive surging, noise and vibration during operation. The engineer designing the spillway should avoid situations where the outlet conduit flows part full and control occurs at the junction of the riser and outlet conduit.

6-1604.1C Riser structures shall be designed with a factor of safety against flotation equal to or greater than 1.3 under any flooding condition. In addition to this criteria, riser structures in wet ponds shall also have a factor of safety against flotation equal to or greater than 1.5 at the normal pool elevation. When the riser is situated in the embankment, the buoyant weight of submerged fill over the footing projection may be considered.

6-1604.2 Vegetated Emergency Spillways. A vegetated emergency spillway is an open channel spillway located adjacent to a dam embankment for the purpose of conveying excess flood flows safely past a dam. They are excavated in natural earth and shall not be located on any portion of the dam embankment fill. Vegetated emergency spillways generally consist of an inlet channel, control section, and exit channel as shown in Plates 49-6 (49M-6), 50-6 (50M-6) and 51-6 (51M-6). Subcritical flow exists in the inlet channel and supercritical flow usually exists in the exit channel. The overland emergency spillway is designed to convey a predetermined emergency spillway design flood without excessive velocities and a predetermined freeboard storm without overtopping the dam embankment.

6-1604.2A Procedures for Vegetated Emergency Spillway Layout:

6-1604.2A(1) The inlet channel should be level for a minimum distance of 20' to 30' (6m to 9m) upstream of the control section.

6-1604.2A(2) The level part of the inlet channel should be the same width as the exit channel.

6-1604.2A(3) Curvature in the inlet channel is acceptable; however, it shall only be introduced upstream of the level section and shall be tangent to the level section.

6-1604.2A(4) The centerline of the exit channel shall be straight and perpendicular to the control section to a point far enough below the earth dam embankment to ensure that any flow which might escape from the exit channel cannot damage the earth dam (see Plates 49-6 (49M-6) and 50-6 (50M-6)).

6-1604.2B Hydraulic Design Procedures for Vegetated Emergency Spillways:

6-1604.2B(1) Vegetated emergency spillways shall be designed to convey the spillway design flood determined in accordance with § 6-1603.

6-1604.2B(2) If a control section is used, then the grade of the exit channel is to be sufficiently steep to ensure supercritical flow (see Plate 51-6 (51M-6)).

6-1604.2B(3) A "Manning's" roughness coefficient (n) of 0.04 is to be used for determining velocity and capacity in vegetated emergency spillways. Uniform flow may be assumed in the exit channel when the flow is supercritical; however, this assumption will be less accurate when the channel slope approaches or exceeds 10%. Where flow is subcritical, such as in the inlet channel, step backwater computations should be performed to determine the head loss between the control section and the reservoir pool when it is apparent that significant head loss may occur through the inlet channel. In cases where the inlet channel is very short and expands rapidly into the reservoir area, a step backwater analysis may not be required; the simple weir formula or direct calculation of critical depth may be used under these circumstances to estimate the energy head upstream of the control section.

6-1604.2B(4) Maximum permissible flow velocities for the SDF in vegetated emergency spillways should be determined in accordance with Plate 52-6 (52M-6).

6-1604.2B(5) The frequency of use of a vegetated emergency spillway should be limited in accordance with § 6-1603.2.

6-1604.3 Riprap Emergency Spillways. A riprap emergency spillway may be required when design velocities exceed those which are acceptable for vegetated emergency spillways.

6-1604.3A The layout of a riprap spillway shall be the same as that described for a vegetated emergency spillway in § 6-1604.2. The hydraulic design of a riprap spillway is similar to that of a vegetated spillway except that roughness coefficients and permissible velocities for riprap shall be used. Assuming dumped riprap placement and side slopes no steeper than 2H:1V, the following roughness coefficients and limiting velocities are applicable:

Riprap Size	Mean Stone Diameter	Roughness Coefficient	Maximum Allowable Velocity
(VDOT Standard)	Ft (mm)	(Manning's "n")	FPS (MPS)
CLASS I	1.1 (350)	0.040	10.0 (3.0)
CLASS II	1.6 (500)	0.043	11.5 (3.5)
CLASS III	2.2 (675)	0.045	13.0 (4.0)

6-1604.3B Riprap shall be placed at a depth equal to twice the mean stone diameter and the terminus of the riprap exit channel shall be keyed into the existing ground to a depth equal to 4 times the mean stone diameter. Riprap bedding shall meet VDOT standards and specifications. The frequency of use of a riprap spillway shall be limited similar to vegetated spillways by setting the crest elevation in accordance with § 6-1603.2.

6-1604.4 The combination of a drop inlet spillway and an overland emergency spillway (vegetated or riprap) is generally required. Standard riser types recommended for drop inlet spillways are shown in Plates 53-6 (53M-6), 54-6 (54M-6) and 55-6 (55M-6).

6-1604.5 Other types of spillways may be considered as alternatives to the drop inlet spillway and overland emergency spillway. Such other types of spillways include: straight drop (free overflow) spillway, ogee crest weirs, side channel spillways, and combined principal/emergency spillways. Topographic and other physical limitations will be considered in determining one type of spillway over another. In no case will an emergency overflow spillway be permitted on the dam embankment fill.

6-1604.6 Combined Principal and Emergency Spillways. As discussed in § 6-1604.4, an emergency spillway separate from the principal spillway is generally required. However, in some cases, it may not be practical to incorporate an overland emergency spillway at either dam abutment due to topographic limitations (e.g., abutments too steep), land use limitations (e.g., existing or proposed development), or some other factor (e.g., roadway embankments acting as dams). In these instances, subject to approval by the Director, a combined principal/emergency spillway may be considered. A combined principal/emergency spillway is simply a single spillway structure which conveys both low flows (e.g., stormwater management functions) as well as extreme flows (e.g., spillway design flood). The combined spillway may take the form of a drop inlet spillway, a straight drop (free overfall) spillway, or some other spillway type. Plates 56-6 (56M-6) and 57-6 (57M-6) depict a combined principal/emergency drop inlet spillway when using a roadway embankment as a dam. A primary design consideration for a combined principal/emergency spillway, particularly when in the form of drop inlet spillways, is protection against clogging. Trash racks shall be designed in accordance with § 6-1604.8. When a drop inlet spillway is proposed as a combined principal/emergency spillway, the SDF and FBH determined in accordance with § 6-1603 shall be routed through the impoundment assuming that no storage is available below the riser crest or rim, and that all ports or orifices along the riser column are inoperative or clogged.

6-1604.7 Stilling Basins. A stilling basin should be incorporated at the downstream end of most spillway structures to help dissipate the high energy flow in the spillway and prevent excessive erosion downstream of the spillway. Stilling basins can take the form of riprap at the endwall of an outlet conduit of a drop inlet spillway or may consist of a sophisticated hydraulic jump basin with impact blocks. The type of stilling basin required is a function of the flow velocities associated with the spillway design flood and the amount of energy dissipation required. Riprap is the preferred form of stilling basin when it can be designed within the parameters and constraints outlined below.

6-1604.7A The following procedure shall be used for designing riprap-type energy dissipators such as those typically placed downstream of outlet conduits discharging through dam embankments. All relevant computations shall be shown on the construction plans.

6-1604.7A(1) Determine the spillway design flood velocity at the outlet.

6-1604.7A(2) From Plate 58-6 (58M-6), determine the size of riprap required to withstand flow velocity.

6-1604.7A(3) If the riprap size selected can withstand the flow velocity in accordance with Plate 58-6 (58M-6), then the riprap should be placed in accordance with the detail in Plate 59-6 (59M-6). The depth of riprap placement should be 2 times D_{50} and the length of the riprap placement should be determined in accordance with the procedures outlined in FHA HEC-14 Chapter XI, where the length of the basin is 15 times the anticipated depth of scour. However, in no case should the length of riprap be less than 4 times the height of the outlet conduit.

6-1604.7A(4) If the riprap size selected cannot withstand the flow velocity in accordance with Place 58-6 (58M-6), then a riprap basin should be designed in accordance with the procedures outlined in FHA HEC-14. In this case, the riprap basin is designed and shaped to reflect the depth of scour that will occur for the size riprap selected. Note that the detail in Plate 59-6 (59M-6) does not apply in this situation. Instead, the detail shown in Plate 60-6 (60M-6) (Figure XI-13 of FHA HEC-14) should be used. It should also be noted that a culvert endwall incorporating wing wall flair should be avoided with this type of basin. Culvert end treatment should be limited to a headwall only and the headwall should extend perpendicular to the pipe alignment. End treatment similar to the VDOT Standard EW-1 is preferred to the Standard EW-2.

6-1604.7A(5) For Froude Numbers greater than 3, some other type of energy dissipator or stilling basin should be investigated such as impact basins or hydraulic jump basins.

6-1604.7A(6) For high tailwater stilling basins, the high velocity core of water emerging from the culvert may retain its jetlike character as it passes through the stilling basin, and be diffused in a manner similar to that of a concentrated jet diffusing in a large body of water. As a result, the scour hole will generally be shallower and longer. Therefore, additional riprap may be required for the channel downstream of the riprap stilling basin.

6-1604.7B Reference is made to the Federal Highway Administration (FHA) publication HEC-14 for design information on several different types of stilling basins and energy dissipators. Also, publications by the Soil Conservation Service, the Bureau of Reclamation , and the Army Corps of Engineers can be referenced for stilling basin design.

6-1604.8 Trash Racks and Debris Control Devices. Most spillways will be subject to some degree of trash and debris associated with incoming flows, and certain spillways are more susceptible than others to clogging as a result of debris. Prior to design of a debris control structure, an assessment of the anticipated debris problem shall be performed. The type and quantity of debris will be largely affected by upstream land use, soil erodibility, watershed size, and the type of stormwater management facility. Generally, debris control structures associated with wet ponds (permanent impoundments) will be designed to protect spillway structures from floating debris, including grass clippings, small limbs, trash, construction debris, logs and trees. Debris control structures associated with dry ponds will generally be designed to protect spillway structures from flowing debris as well as floating debris. Flowing debris may include silt, sand, gravel, trash, rock fragments, and construction debris; all of which may be transported as a bedload of the flood flow. The following design criteria shall be used:

6-1604.8A Trash racks for tops of drop inlet spillways should be designed to protect against clogging of the spillway under any operating level. The average velocity of flow through a clean trash rack should not exceed 2.5 FPS (0.75 MPS) for operation during the spillway design flood. Velocity can be computed on the basis of the net area of opening through that portion of the rack experiencing flow. This same criteria shall also apply to ports or openings along the side of a riser structure. Bar spacing should be no greater than 1/2 of the minimum conduit dimension in the drop inlet spillway, and, to discourage child access, bar spacings shall be no greater than 1' (300mm) apart. The clear distance between bars shall generally not be less than 2" (50mm); however, one exception to this may be near the apex of the trash rack shown in Plate 53-6 (53M-6).

6-1604.8B Debris control devices for dry stormwater management ponds may be required for low level intakes at the pond bottom. In these situations, debris control structures such as those discussed in the FHA publication entitled "Debris Control Structures (HEC No. 9)" should be considered where appropriate.

6-1604.8C Debris control devices for extended dry stormwater management facilities are required for the low flow orifice controlling the extended drawdown period. The preferred trash rack detail for those facilities is shown in Plate 61-6 (61M-6).

6-1604.8D In some cases, for both wet and dry stormwater management facilities, debris racks such as those discussed in FHA HEC No. 9 may be required at major inflow locations to the stormwater management basin so that debris can be intercepted prior to entering the basin.

6-1604.8E Recommended debris control devices for riser structures are contained in Plates 53-6 (53M-6) through 55-6 (55M-6).

6-1604.9 Anti-Vortex Devices. All closed conduit spillways designed for pressure flow shall have adequate anti-vortex devices. Anti-vortex devices may take the form of a baffle or plate set on top of a riser, or a headwall set on one side of a riser. The SCS 2-way covered riser (see example detail on Plate 55-6 (55M-6)) has very reliable anti-vortex and debris control provisions inherent in the standard design. Example details for some recommended anti-vortex devices are shown on Plates 53-6 (53M-6) through 55-6 (55M-6).

6-1604.10 Drain Valves. Stormwater management facilities having permanent impoundments (i.e., wet ponds) shall be designed to allow draining the permanent pool to facilitate lake maintenance and sediment removal. The draining mechanism will usually consist of some type of valve or gate attached to the spillway structure. The following design guidelines for pond drains are provided:

6-1604.10A Pond drains shall be designed with sufficient capacity to pass a flood having a 1-yr recurrence interval with limited ponding in the reservoir area such that sediment removal or other maintenance functions are not interrupted. The pond drain system may be no smaller than 8" (200mm) in diameter.

6-1604.10B Pond drains shall be designed with adequate trash racks or debris control devices. Trash racks should generally be designed such that flow velocities through the rack are less than 2 FPS (0.60 MPS). However, velocities of up to 5 FPS (1.5 MPS) may be allowed for racks that are easily accessible for cleaning. The spacing of trash rack bars shall depend on the size of the outlet conduit or valve and the type of valve or gate. When a pond drain consists of a small conduit with a valve control, closely spaced trash bars may be required to exclude small trash. When large conduits with sluice gates are used, the trash bars can be more widely spaced. In general, trash bars may be placed 3" to 6" (75mm to 150mm) apart and assembled in a grid pattern.

6-1604.10C In most cases, sluice gates are preferred over "in-line" type valves such as those used in water distribution systems (e.g., eccentric plug valves, knife gate valves, gate valves). Sluice gates are generally more appropriate for passing debris-laden flow, are generally less prone to clogging, and are usually easier to maintain. Standard sluice gate nomenclature is provided on Plate 62-6 (62M-6).

6-1604.10D An uncontrolled or rapid drawdown could induce problems such as slides in the saturated up-stream slope of the dam embankment or shoreline area. Therefore, the design of a pond drain system shall include operating instructions regarding draining the impoundment. Generally, drawdown rates shall not exceed 6" (150mm) per day. For dam embankments or shoreline slopes of clay or silt, drawdown rates as slow as 1' (300mm) per week may be required to ensure slope stability.

6-1604.11 Concrete Low Flow Channels (Trickle Ditches):

6-1604.11A When the pond storage area is left in a natural condition and in a conservation easement granted to the Board of Supervisors or Fairfax County Park Authority, a trickle ditch will be provided from the outlet riser to the limits of clearing and grading. This ditch shall be a minimum of 20' (6m) in length and transition from the outlet riser to the natural channel. A concrete or riprap approach shall be installed to direct the flow into the trickle ditch from the natural channel.

6-1604.11B Adequate drainage features shall be provided to enable excavated dry or extended dry pond bottoms to be readily maintained by heavy equipment. Springs or seeps shall be controlled and outfall to the drainage channel. A trickle ditch will normally be installed from a principal inlet to the outlet riser. A concrete or riprap approach shall be installed to direct the flow into the trickle ditch from the principal inlet. Special considerations with pond design should be given for unusual conditions such as extreme length of graded channel, or highly erosive or unstable soils.

6-1604.11C All trickle ditches shall conform to the design set forth in Plate 63-6 (63M-6). The design hydraulic capacity of the ditches shall be the greater of the low flow orifice or that of the minimum size ditch shown in Plate 63-6 (63M-6).

6-1605 Geotechnical Design Guidelines for Stormwater Management Reservoirs with Earthdams

6-1605.1 Introduction:

6-1605.1A Purpose and Scope. The purpose of these geotechnical guidelines is to provide minimum recommended procedures for exploration and minimum requirements for planning and designing stormwater management reservoirs with earthdams. The guidelines are intended to provide the basis for geotechnical design of these facilities. The designer is responsible for determining those aspects of the guidelines that are applicable to the specific facility being designed in addition to satisfying the minimum requirements as provided under § 6-1605 et seq. These guidelines are not intended for use in designing concrete or roller-compacted concrete dams.

6-1605.1B Facility Type. Three general types of facilities are acceptable: dry detention reservoirs, extended dry detention reservoirs and reservoirs with permanent pools. Plate 64-6 (64M-6) shows the minimum level of the geotechnical design guidelines associated with each type of facility.

6-1605.1C Dam Types. These guidelines are for design of earthfill embankment dams. Typically, these dams are designed as homogeneous dams with or without internal drainage. Zoned dams may be designed if the quality of the borrow material requires this approach. Typical embankment dam cross-sections are shown on Plates 65-6 (65M-6) and 66-6 (66M-6).

6-1605.2 Geotechnical Engineering Design Study:

6-1605.2A Submission Requirements. Submission of geotechnical reports to DPWES are required for all reservoirs with permanent pools and for dry and extended dry detention reservoirs in Categories B and C as defined in Plate 64-6 (64M-6). This requirement may be waived by the Director provided the geotechnical aspects of the design are adequately addressed on the grading plans or construction plans.

6-1605.2B Geotechnical Engineer Qualifications. Geotechnical studies shall be performed under the direction of a qualified geotechnical engineer licensed as a Professional Engineer in the Commonwealth of Virginia. The geotechnical engineer must have experience in the design and construction monitoring of dams of the size and scope covered by these guidelines.

6-1605.2C Study Content. The Geotechnical Engineering Design Study shall consist of: (1) field investigation; (2) laboratory testing; and (3) geotechnical engineering analysis. The study must provide the designer with adequate recommendations for the design of the dam, reservoir and appropriate structure.

6-1605.2C(1) Field Investigation. The field investigation program shall be performed to explore the subsurface conditions for the proposed embankment dam, reservoir and borrow area. The field investigation program must include: (1) review of available data; (2) field reconnaissance; and (3) subsurface exploration. Existing information such as topographic and geologic data should be reviewed. References such as soil maps, the General Ratings for Dams, Embankments and Reservoirs (Table 6.27 following § 6-1605.6F(2)), and any other sources of information should be reviewed. This review of available data should be followed by a field reconnaissance of the site of the dam and reservoir. The subsurface exploration program, consisting of test borings, test pits, or both, should be developed based on the complexity of the geologic and topographic features disclosed by the previous phases. Except when adequate measures are taken to restore the natural condition of excavations, test pits shall be in areas outside the alignment of the dam. At a minimum, 3 test borings shall be located along the dam alignment (centerline) and along the principal spillway profile at intervals not to exceed 100' (30m). Additional borings shall be required at each major structure. Borings also shall be required throughout the ponding area at a density of at least 1 per acre (0.4 ha) (evenly distributed) with a minimum of 2 borings for ponding areas less than 2 acres (0.8 ha). The ponding area shall be defined as that area inundated by the 2-yr water surface elevation. The depth of borings shall extend to competent material or to a depth equal to the lesser of either the embankment height or the foundation width. The use of geophysical techniques where applicable is encouraged. The subsurface exploration program shall be designed and implemented to evaluate the foundations, abutments, reservoir area and embankment design and any other pertinent geological considerations. Insitu testing, such as permeability tests, undisturbed sampling and installation of piezometers may be required depending upon the site conditions and anticipated designs.

6-1605.2C(2) Laboratory Testing. Laboratory tests to characterize the various borrow materials and foundation soils are required. At a minimum, an index property test shall be performed to properly classify soils in accordance with the Unified Soil Classification System. Shear strength, compressibility, and permeability testing may be required depending upon the size and complexity of the dam and the nature of the site subsurface conditions.

6-1605.2C(3) Geotechnical Engineering Study. After completion of the field investigation, associated testing and analysis, a report shall be prepared by the geotechnical engineer to present findings, recommendations and comments on items outlined in the design guidelines given in § 6-1605.2C(3) through § 6-1605.6. At a minimum, the report shall include:

6-1605.2C(3)(a) A site location map

6-1605.2C(3)(b) A boring/test pit location map

6-1605.2C(3)(c) A description of the site

6-1605.2C(3)(d) A description of the proposed dam

6-1605.2C(3)(e) Soil and rock strata descriptions to include boring/test pit logs and subsurface profiles

6-1605.2C(3)(f) Geologic characterization of soils and bedrock

6-1605.2C(3)(g) Summarization and discussion of laboratory test results

6-1605.2C(3)(h) Geotechical engineering analysis and recommendations, including foundation preparation/treatment, design of interior drainage features and filters, geotechnical design of conduits/structures through embankment, embankment design including seepage and stability analysis, and important construction considerations

6-1605.2C(3)(i) A description of the subsurface exploration procedures utilized

6-1605.2C(3)(j) A description of laboratory test procedures utilized

6-1605.2C(3)(k) A description of any computer-aided stability analyses utilized

6-1605.2D This report shall be submitted with required construction plans to DPWES for review and approval. The recommendations in the approved report shall be incorporated into the construction plans as requirements to be fulfilled during construction. In addition, the geotechnical engineer shall review all applicable construction plans and provide a statement on the plans that the plans have been prepared in accordance with his recommendations.

6-1605.2E Special Cases:

6-1605.2E(1) Rehabilitation of Existing Dams and Retrofitting Existing Highway Embankments for Use as Dams. The following measures shall be taken when rehabilitating existing dams or when retrofitting existing highway embankments for use as dams for both wet and dry stormwater management facilities:

6-1605.2E(1)(a) A study shall be performed to thoroughly investigate the soils within the existing embankment, and, in the case of existing highway embankments, evaluate potential seepage through the bedding material of existing utilities in the embankment.

6-1605.2E(1)(b) A field investigation which should disclose such items as pervious layers which will allow excessive seepage and piping, inadequate clearing, grubbing and stripping of subgrade, highly variable compaction of materials and animal burrows.

6-1605.2E(1)(c) Sufficient laboratory and insitu testing should be performed to characterize the soils.

6-1605.2E(1)(d) Based upon the investigation and analysis, a report shall be prepared by the geotechnical engineer and submitted to the design engineer and DPWES along with construction plan submittal, presenting design recommendations as described in § 6-1605.2 et seq.

6-1605.2E(2) Raising Height of Existing Dam. A study must be performed to thoroughly investigate the soils of the existing embankment. Sufficient laboratory and insitu testing shall be performed to characterize the soils. Based upon the evaluation and required analysis, recommendations should be prepared to specify material and compaction requirements, acceptable slope, and benching when the method of addition of material to the downstream or upstream slopes is used. The evaluation shall include the consideration of the effect of additional surcharge loads imposed by the added materials, and the effect on any existing utility lines within the embankment. If a flood wall is proposed, the design should include maximum allowable bearing pressures, estimated settlement, footing cover, and lateral earth pressures for use in design. Based upon the investigation and analysis, a report shall be prepared by the geotechnical engineer and submitted to the design engineer and DPWES along with construction plan submittal, as described in § 6-1605.2

6-1605.3 Embankment Dam Foundations:

6-1605.3A Clearing, Grubbing and Stripping. Clearing consists of the removal of all unwanted materials from the foundation area which may create any obstruction or other undesirable design or construction situation. Materials such as trees, bushes, fallen trees, boulders, rubble, garbage, buildings and similar debris must be removed from the foundation area. Grubbing is the removal of all unwanted materials which lie below the ground surface in the foundation area including stumps, roots, drain fields, abandoned utility lines, foundations, and other materials or buried structures. Stripping is the removal of topsoil, organic matter, excessively soft soil and any other deleterious materials. The limits of clearing, grubbing and stripping in the dam foundation and abutment area shall extend at least 10' (3m) beyond the toe of the slope of the dam embankment. Excavations in the dam foundation area shall be properly backfilled with embankment material as defined in § 6-1605.6A through § 6-1605.6F. The clearing limits for overland emergency spillways shall extend at least 10' (3m) beyond the top of the cut slope. Clearing limits for maintenance access roads shall extend at least 5' (1.5m) beyond the edge of pavement. Clearing in excess of the minimum limits outlined above is permitted when necessary; however, designers are encouraged to maintain the minimum clearing limits to the extent practical.

6-1605.3B Stream Diversion. The design of most dams should include the provision for streamflow diversion around or through the dam site during the construction period. Streamflow diversion can be accomplished by numerous acceptable means, including open channels, conduits, cofferdams and pumping. In most cases, stream diversion is accomplished in 2 phases. The first phase usually involves diversion of the stream into a man-made open channel which will convey drainage around the majority of the dam site. The second phase usually involves redirection of flow from the man-made diversion channel into a low flow conduit passing through the dam. At a minimum, the capacity of the diversion system shall be adequate to safely pass a 2-yr recurrence flood. Designers are encouraged to increase capacity if it is likely that serious or costly damage may result if the capacity of the diversion system is exceeded. The diversion system shall be protected against erosion during the 2-yr design storm through the use of appropriate channel linings, riprap drop structures or other suitable measures.

6-1605.3C Foundation Design (Treatment). The dam foundation includes the entire stream valley and abutments covered by the embankment. The objectives of foundation surface treatment are (1) embankment foundation bonding, (2) preventing piping of embankment material, (3) mitigating adverse impacts resulting from the presence of unsuitable foundation materials, and (4) preventing embankment cracking.

6-1605.3C(1) Foundation Suitability. The suitability of the foundation for support of the embankment is determined by the ability of the foundation to support the embankment (1) without detrimental settlement and associated embankment cracking, and (2) without excessive seepage which could cause excess loss of reservoir water or, in severe cases, piping and embankment failure. The suitability of the soils to support the embankment loading and control seepage should be determined during the geotechnical engineering design study and included in the report described in § 6-1605.2A through § 6-1605.2D.

6-1605.3C(1)(a) Compressibility. Soils which are highly compressible and which will cause excessive total and differential settlement, such as soft organic soils, should be undercut prior to embankment construction. Soils to be undercut must be identified and defined in the geotechnical engineering design study and the actual extent of undercut must be verified in the field by the geotechnical engineer during construction.

6-1605.3C(1)(b) Seepage. To determine how to control foundation seepage, both soil and rock properties should be considered depending upon the geology of the site and the hydraulic head. Within a zoned or a homogeneous embankment dam, a cutoff trench to rock or to an impervious soil stratum is usually employed to control seepage through the foundation. Partially penetrating cutoff trenches should not be used solely for the purpose of reducing seepage. Such trenches may, however, be employed for stability purposes. Alternative seepage control measures such as upstream synthetic membranes, a central diaphragm such as a soil bentonite or cement bentonite slurry wall, or an upstream impervious blanket, may also be used for seepage control, as described in § 6-1605.6.

6-1605.3C(2) Cutoff Trench. The width (W) of the cutoff trench should be equal to the height of the reservoir (h), less the depth below the ground surface to the impervious soil or rock Stratum (d) or W=h-d, with a minimum width of 8' (2.4m). In order to obtain adequate compaction against the sides of a cutoff trench, the trench shall have sloping sides as described in § 6-1605.3C(3) and § 6-1605.3C(4).

6-1605.3C(3) Rock Foundations. Rock foundations require special treatment to provide a proper bond between the foundation and the embankment material as described below.

6-1605.3C(3)(a) Slopes. Rock slopes should not be greater than 0.5H:1V. All overhangs must be removed. Within the core zone or cutoff trench, slopes steeper than 0.5H:1V shall be excavated or treated with dental concrete if they are greater than 1' (0.3m) high. In other foundation areas, the height of slopes steeper than 0.5H:1V should not exceed 5' (1.5m).

6-1605.3C(3)(b) Blasting. Generally, no blasting shall be permitted within 100' (30m) of the dam foundation and abutment area. If blasting must be performed, it shall be carried out under controlled conditions to reduce adverse effects on the rock foundation, such as overblasting and opening fractures. Blasting should be performed by a specialty contractor experienced in blasting techniques. Blasting procedures shall be submitted to the geotechnical engineer for review prior to use.

6-1605.3C(3)(c) Surface Cleaning. Within the cutoff trench area, the surface of the rock must be cleaned of all objectionable material by hand work, brooming , or by air or water jetting. Loose or unsuitable material must be removed from all cracks, seams or shear zones to a depth of 3 times the width of the feature up to 5' (1.5m) in depth, and as determined in the field in the case of wider features.

6-1605.3C(3)(d) Dental Concrete. Cracks which have been cleaned, areas into which it would be difficult to compact soil, and other uneven features such as overhangs and steep slopes, must be filled with dental concrete. Dental concrete shall have mix proportions to ensure a 28-day strength of at least 3000 PSI (20.7 mPa). The rock surface should be thoroughly cleaned and moistened prior to placement of dental concrete to ensure a proper bond. The surface of the dental concrete should be broom finished to assure proper bond with the overlying soil. A minimum curing time of 24-hr under curing conditions approved by the designer must be provided prior to placement of soil fill material.

6-1605.3C(3)(e) Slush Grouting. Slush grout consisting of "neat" cement and water, or cement, sand and water, should be used to fill small cracks. The maximum aggregate size should not exceed 1/3 the crack width. Gout shall be used within 30 minutes of mixing.

6-1605.3C(3)(f) Grout Curtain. Grouting of foundation rock may be required depending upon the site geology, fracture condition of the rock and the reservoir head conditions. Generally, the grout curtain should extend to a depth below the foundation and abutment level equal to the reservoir head above the location. Laterally, the curtain should extend at least to the end of the abutments. If grouting is required, a minimum double line grout curtain should be used. Primary grout hole spacing of 25' to 40' (7.5m to 12m) is typical with staggered hole locations between the 2 grout hole lines. The spacing between lines is typically 10' (3m). Secondary grout holes should split the spacing between primary holes. The need for tertiary holes will depend upon the grout take from the secondary holes. Grout pressures will depend upon site conditions. Generally, grout pressures should not exceed 1 PSI per ft (22.6 kPa per m) of depth below the surface. A concrete grout cap should be employed if numerous fractures intersect the surface of the rock. Grout hole spacing and depth, grout mix design and consistency and grout pressures should be in accordance with current grouting practice design.

6-1605.3C(3)(g) Filters. The downstream surface of the cutoff trench shall be evaluated for the potential of piping of embankment material into the foundation soil or rock. Filters should be designed in accordance with the criteria included in § 6-1605.4.

6-1605.3C(3)(h) Earth Fill Placement. Earth fill placed within the cutoff trench and the remainder of the embankment must be placed and compacted according to the criteria included in § 6-1605.6F.

6-1605.3C(4) Soil Foundations. The following special conditions are applicable to the preparation of soil foundations to receive embankment materials.

6-1605.3C(4)(a) Excavation and Shaping. Surface irregularities should be removed to provide satisfactory foundation contours. Slopes should be sufficiently flat to prevent sloughing, but in no case greater than 1H:1V. Disturbed materials must be removed to a depth of at least 6" (150mm).

6-1605.3C(4)(b) Subgrade Preparation. The subgrade must be compacted to a depth of at least 6" (150mm) to the required density standard as defined in § 6-1605.6 of these Design Standards. Fine-grained subgrades should be scarified to a depth of at least 6" (150mm) prior to compacting. Coarse-grained subgrades generally should not be scarified. Proofrolling is usually performed to locate soft unsuitable surficial soils; however, proofrolling may not be feasible if soils are too soft for heavy equipment, or if a firm layer exists over a soft layer in which case proofrolling could cause deterioration of the subgrade.

6-1605.3C(4)(c) Foundation Dewatering. The foundation shall be dewatered to ensure that the surficial 6" (150mm) of the subgrade to be compacted is not saturated and pumping of the subgrade does not occur under the weight of compaction equipment. Pumping from sumps, well points, or deep wells may be required for excavation of the cutoff trench.

6-1605.4 Embankment and Foundation Seepage Control. The control of seepage through its embankment is required for all dams. Leakage through a reasonably well-constructed embankment occurs from: (1) cracks due to shrinkage; (2) cracks due to differential settlement; and (3) cracks due to hydraulic fracturing in zones of reduced stress. Filters are effective in controlling erosion resulting from seepage. Seepage may be controlled with: (1) a downstream toe drain; (2) a downstream drainage blanket; or (3) a chimney drain along the downstream side of the core of a zoned embankment dam or within a homogeneous earth fill dam. Appropriately designed filters must be provided between the drainage materials and the soil foundation or embankment. An area of special concern is the control of seepage along conduits or structures penetrating the embankment. Cutoff collars or other protruding features should not be used solely for seepage control. Seepage along the conduit or structure through the embankment foundation and/or "impervious" zone should be controlled by the use of properly designed filters and drainage features in the downstream portion of the embankment.

6-1605.4A Filter, Drainage Layer and Toe Drain Design. Filters should be designed to prevent migration of fines from the foundation or embankment soils into the drainage layer. The filter should be designed for stabilizing migration of the soil into the filter as follows:

SOIL TYPE	FILTER CRITERIA
SILTS & CLAYS (> 85% Passing No. 200 sieve)	$D_{15F}/D_{85B} = < 9$
SANDY SILTS AND CLAYS SILTY AND CLAYEY SANDS (40 to 85% Passing No. 200 sieve)	$D_{15F} < 0.7$mm
SANDS AND SANDY GRAVELS (0 to 40% passing No. 200 sieve)	$D_{15F}/ D_{85B} = < 4$

6-1605.4A(1) (57-96-PFM) The filter should be designed for adequate permeability to drain the soil as follows:

D_{15F}/D_{85B} less than or equal to 5

6-1605.4A(2) D_{15F} is the size of the filter material for which 15% is finer and D_{85B} is the size of the base material being drained for which 85% is finer.

6-1605.4A(3) The percentages of base material passing the No. 200 sieve pertains to the fraction excluding material retained on the No. 4 sieve. The drainage layer and toe drain piping system should be sufficiently porous and sized to accommodate the anticipated flow of water into the drainage system. The minimum design requirements for embankment and foundation seepage control design are set forth on Plate 64-6 (64M-6).

6-1605.4B Geosynthetics. Non-woven geosynthetic fabrics and drainage nets with non-woven geosynthetic fabric facings (geonets) may be used in place of sand filters to control seepage through earthfill embankments under restricted conditions. Restrictions are required due to the lack of data concerning the effectiveness of these products to transmit seepage without clogging over extended periods of time under steady seepage conditions.

6-1605.4B(1) Geosynthetic Fabrics and Geonets. These materials should not be used as filters under steady seepage conditions with greater than 6' (1.8m) of head. This applies to horizontal drainage blankets or drainage blankets surrounding the downstream portion of the principal spillway pipe, or as filters between the "impervious" fill of a cutoff trench and the downstream face of the cutoff trench. They may be used as filters surrounding drainage materials in downstream trench type toe drains. They may also be used as filters under non-steady seepage conditions, such as for dry and extended dry detention reservoirs.

6-1605.4B(2) Geosynthetic Design. The design should consider permeability and pore size in selecting geosynthetics for use as filters. Non-woven and woven geosynthetic fabrics may also be used for erosion control in place of graded filters beneath riprap on grout type mattresses. The design should consider permeability, pore size and hydraulic gradient in selecting a geosynthetic. Design procedures published by any Federal agency (e.g., COE, SCS) as well as those recommended by the manufacturer of the geosynthetic are generally acceptable. Geosynthetics may be used for material separation.

6-1605.5 Design of Conduits/Structures through Embankments. Because the contact between the soil embankment, the foundation material and the embankment penetrating conduits is the most susceptible location for piping, special attention must be given to the design of any conduit penetrating a dam embankment. The number of these conduits shall be minimized, and whenever possible, utility conduits other than the principal spillway should be located outside of the dam embankment. All conduits penetrating dam embankments shall be designed in accordance with the following criteria:

6-1605.5A Conduit/Structure. Conduits and structures penetrating the embankment should have a reasonably smooth surface and should not have protrusions or indentations that will hinder compaction of embankment materials.

6-1605.5A(1) Shape. Cast-in-place conduits should be formed such that the side surfaces slope at 1H:10V to facilitate compaction of soil against the conduit in the impervious zone and the embankment upstream of the drainage blanket. Within the drainage blanket, vertical surfaces are acceptable. Where pipe is to be used as a conduit, a concrete cradle shall be provided such that the resulting surface of the conduit has a slope of 1H:10V up to the level of the spring line of the pipe. A standard concrete cradle detail is provided in Plate 67-6 (67M-6). Cradles on yielding foundations should be articulated. Cradle requirements depend upon dam classification as noted in Plate 64-6 (64M-6). The length of the cradle generally shall extend from the riser structure to the beginning of the seepage collection zone. Other structures penetrating the embankment, such as cast-in-place concrete sections of the dam, should also have battered side surfaces of 1H:10V upstream of the drainage layer.

6-1605.5A(2) Pipe. Principal spillway pipe shall be reinforced concrete pipe meeting the specifications of § 6-1607.1B(4). The minimum allowable pipe diameter shall be 18" (450mm).

6-1605.5A(3) Pipe Joints. Pipe joints shall be designed to remain watertight during the life of the structure under maximum anticipated hydrostatic head and maximum likely joint opening related to foundation settlement. Round rubber gaskets set in a groove are required for all precast concrete pipe conduits.

6-1605.5A(4) Rock Foundation Preparation. The surface of rock for support of the conduit/structure should be prepared as stated in § 6-1605.3C.

6-1605.5A(5) Backfill Below Rock Surface. Cast-in-place conduits/structures and pipes founded below the rock surface in trenches upstream of the drainage blanket should be backfilled with dental concrete to the surface of the rock as set forth in § 6-1605.3C. Alternatively, the structure may be cast against the surface of the rock within this zone. Nonstructural concrete backfill or casting the conduit/structure against the surface of the rock may also be used throughout the embankment section upstream of the drainage zone. When a pipe is used as a conduit, a concrete cradle shall be provided. The cradle should be cast surrounding the pipe to the level of the spring line of the pipe. The cradle may be cast against the rock surface. Soil backfill as stated in § 6-1605.6F, may be used above the rock surface for all types of structures.

6-1605.5A(6) Soil Foundation Preparation. The soil subgrade for support of the conduit/structure should be prepared as set forth in § 6-1605.3C.

6-1605.5A(7) General Earthfill Requirements. Earthfill adjacent to conduits/structures should be placed so that lifts are at the same level on both sides of the conduit. In order to improve the quality of compaction adjacent to the conduit/structure, fill may be ramped away from the conduit/structure on a 6H:1V slope. Compaction equipment must be approved by the geotechnical engineer. Material quality, moisture content, lift thickness and compacted density should be the same as required for other similar portions of the embankment or impervious zone. Use of hand compaction is not recommended. When hand-type equipment is used, the maximum lift thickness shall not exceed 4" (100mm). General criteria for backfill soils are included in § 6-1605.6F.

6-1605.5B Seepage Control Along Embankment Conduits/Structures. The contact between the soil embankment and foundation material and conduits/structures which penetrate an embankment are the most likely locations for piping. This is due to the discontinuity formed by the contact and the difficulty in compacting soil in this zone. Collection and control of this seepage by filters and a drainage system is required. The seepage control system or drainage blanket provided for the conduit/structure should be contiguous with the drainage blanket placed within the embankment; otherwise a separate drainage system should be provided.

6-1605.5B(1) Drainage Blanket. The drainage blanket should completely surround the conduit/structure if supported on soil, or should extend above the rock surface of the conduit/structure cradle/foundation if supported on rock. In the case of a homogeneous dam, the drainage system should extend from the downstream toe to the downstream point 1/3 of the distance of the base width. In the case of a zoned dam, the drainage system should extend from the downstream side of the core to the downstream toe of the dam. If a drainage blanket is used, the conduit/structure filter should extend to the upstream edge of the blanket, or at a minimum, to the downstream point 1/3 the distance of the base width of the dam. The filter and drainage blanket should be designed in accordance with the criteria in § 6-1605.4A through § 6-1605.4B. The requirements to determine for the necessity of drainage blankets adjacent to conduits/structures are stated in Plate 64-6 (64M-6).

6-1605.6 Dam Embankment Design:

6-1605.6A Geometry. Dam embankments typically are dams constructed as homogeneous (with or without internal drainage), zoned or diaphragm-type structures. Homogeneous dams without internal drainage are not permitted for reservoirs with permanent pools if the phreatic surface will intersect the downstream surface of the dam. Homogeneous dams without internal drainage may be permitted for reservoirs with permanent pools if justified by an approved seepage analysis. However, for dry detention or extended dry detention (BMP) reservoirs where a saturated condition does not exist, a homogeneous dam without internal drainage may be appropriate. Thin diaphragms, such as soil bentonite slurry walls, may also be used within homogeneous sections to control seepage.

6-1605.6A(1) Height. The height of a dam embankment shall be based upon the freeboard (see § 6-1603), wave action and compensation for settlement, and the design capacity of the pond.

6-1605.6A(2) Crest. The crest of a dam embankment should be designed with the following considerations:

6-1605.6A(2)(a) Width. The minimum top width may be determined by the following equation, but shall not be less than 12' (3.7m):

$$W = (H + 35)/5$$
$$(W = (H + 10.7)/5)$$

W = width of crest, ft (m)
H = height of dam above downstream toe at stream bed, ft (m)

6-1605.6A(2)(b) Drainage. Surface drainage should be provided by either crowning or sloping towards the upstream slope. The minimum slope is 2 %.

6-1605.6A(2)(c) Camber. Camber is provided to maintain the height of the dam lost due to compression of the foundation soils or soils within the embankment. Camber will be based on the estimated total compression and will vary from the abutments to the center.

6-1605.6A(2)(d) Surfacing. A grass surface is permitted unless frequent vehicular traffic or foot travel is expected, in which case a gravel or bituminous surface shall be required to prevent erosion.

6-1605.6A(2)(e) Safety Requirements. Crests that are used for roadways within the state right-of-way shall be provided with guard rails or other safety devices in accordance with VDOT standards.

6-1605.6A(3) Non-linear. Dam embankments should be linear. If a non-linear section is proposed, the shape should be limited to concave upstream geometry.

6-1605.6B Zoning. Zoning, if required, should consist of impervious materials on the upstream side and pervious materials on the downstream side. If impervious material is scarce, then an impervious core or a thick blanket of impervious material on the upstream embankment face may be considered.

6-1605.6B(1) Impervious Core Thickness. The impervious core thickness design shall take into account tolerable seepage loss, minimum width which will permit proper construction, type of material available for core and shells, and the design of proposed filters. Suggested minimum and maximum core sizes are set forth in Plate 68-6 (68M-6). The core sizes given in Plate 68-6 (68M-6) are suggested and other core thickness may be considered. However, dams with cores smaller than minimum Core Size A should be designed as a diaphragm-type and cores larger than Core B should be designed as homogeneous. Recommendations for the core should include material type, compaction, filter requirements, width, height and side slopes.

6-1605.6C Stability Analysis. The design of slopes for dam embankments depends on the materials used for construction , foundation conditions, height of the embankment, pool level and whether the embankment is for permanent storage (wet ponds) or detention (dry or extended dry detention ponds).

6-1605.6C(1) Slope Design by Material Type. Plate 69-6 (69M-6) states the maximum upstream and downstream slopes for dam embankments with various soil types in cases when detailed seepage and stability analyses are not required. The slopes in these tables are for stable foundations. If complicated foundation conditions exist or only poor quality construction materials are available, then a detailed stability analysis should be performed.

6-1605.6C(2) Stability Analysis. When required in accordance with Plate 64-6 (64M-6), slope stability analyses should be based on the criteria specified in SCS TR-60. These analyses require determination of shear strength parameters developed for site specific conditions and materials to be used in construction of the dam. The County is located in seismic zone 2, and as such, dams must be designed for a seismic coefficient of 0.05. The factor of safety for the various conditions is calculated based on the factor of the shear strength available to the shear strength mobilized. Clear documentation of assumptions, conditions analyzed and not analyzed, and correlated shear strength parameters is required. Calculations must be submitted with the design to DPWES.

6-1605.6C(3) Raising Height of Existing Dam. Raising the height of an embankment dam may be performed by adding material to the upstream slope, downstream slope or both. Raising the height by adding material to the slopes should be constructed as an integral part of the existing embankment. Recommendations for material type, moisture content, compaction requirements and side slopes should be provided. Embankment slopes should be determined by seepage and stability analyses or be based upon material type depending upon the reservoir classification in Plate 64-6 (64M-6).

6-1605.6D Upstream Blankets. An upstream soil blanket consisting of material similar to the homogeneous dam may be used to reduce seepage through a pervious foundation if the material is sufficiently impervious. The thickness of the blanket will be influenced by acceptable seepage loss, permeability of blanket and foundation dam material, reservoir head, unsaturated or saturated foundation, and constructability. Blankets must be no less than 1.5' (450mm) thick. An impervious upstream soil blanket may also be used for a homogeneous dam constructed of more pervious soils or with a zoned embankment. The thickness, material, and compaction requirements for soil blankets should be specified and migration of fines into the foundation must be evaluated. Where filters are required, they should be designed in accordance with § 6-1605.4A through § 6-1605.4B. For synthetic liners, material type, subgrade preparation, bedding, seam preparation and overlap should be specified.

6-1605.6E Cutoff Trench. The cutoff trench shall be backfilled with relatively impervious material for homogeneous dams or the impervious core material for a zoned dam. Recommendations for backfill of the cutoff trench shall include soil type, moisture content and compaction requirements.

6-1605.6F Compacted Fill Requirements:

6-1605.6F(1) Soil Type. The soil type shall be specified by use of the Unified Soil Classification System. Rock fill types shall be specified by maximum or minimum percent passing various sieve sizes and durability to resist deterioration from slaking or weathering.

6-1605.6F(2) Compaction. Compaction requirements should be specified. The requirements should include the percent of maximum dry density for the specified density standard, allowable range of moisture content, and maximum loose lift thickness. Dam embankment compaction to 95% of the maximum dry density in accordance with ASTM D-698 or AASHTO T-99 is considered a minimum standard. The allowable moisture content for compaction will vary depending on material type. Generally, the moisture content specified shall take into consideration the permeability of embankment material, the potential for expansion, shrinkage, and cracking, and shall permit uniform compaction to the project specification without any yielding of the fill surface. Fill must be placed in layers with a maximum thickness which will allow uniform density throughout the compacted layer. For materials with more than 5% passing the 200 sieve, the maximum loose thickness should generally be between 6" and 8" (150mm and 200mm). Thicker loose lifts may be specified, but in no case shall the loose lift thickness exceed 12" (300mm). Lift thickness for open-graded stone or gravel may vary from 12" to 24" (300mm to 600 mm), with 12" (300mm) being the minimum for hand equipment and 24" (600mm) being the maximum for large vibrating rollers. Any layer of fine grained fill which becomes smooth under compaction or construction traffic should be scarified to a depth of 2" (50mm) to allow adequate bond between layers. Initial lifts for the cutoff trench on rock foundations must be placed and compacted by methods that will assure adequate compaction without damaging the rock surface. This can be accomplished by making the initial lift thickness 150% of the specified normal lift thickness and then stripping the upper 50% after initial compaction and recompacting the lower portion to specification.

TABLE 6.27 GENERAL RATINGS FOR DAMS, EMBANKMENTS AND RESERVOIRS (56-96-PFM)

No.	Soil Name[1]	Physiographic Province/ Parent Material/ Landscape Position[2]	Typical USCS Classification[3]	Embank-ment Materials[4]	Embankment Foundation[4]	Core/Liner Materials[4]	Seepage Potential[5]	Erosion Potential[6]
1	Mixed Alluvial	(Tr, Pd, Cp) Silty, sandy, and clayey recent alluvium in floodplains	Variable – CH to GM	Marginal – W, P, O	Poor - B, W, O	Marginal – W, P, O	Moderate	Low
2	Chewacla	(Pd) Silty alluvium on low ter-races in floodplains	ML	Marginal – W, P	Poor – B, W	Marginal – W, P	Moderate	Low
3	Congaree	(Pd) Silty alluvium on low ter-races in floodplains	ML	Fair – P, W	Marginal – B, W	Fair – P, W	Moderate	Low
5	Wehadkee	(Pd) Silty and clayey alluvium on low terraces in floodplains	CL, MH, ML, CH	Marginal – W, P	Poor – B, W	Marginal – W, P	Low	Low
6	Hyattsville	(Cp) Silty to sandy local allu-vium overlying Coastal Plain sediments	CL, SM, SC	Fair – P, W	Fair – B, W	Marginal – T, P, W	Moderate	Low
8	Worsham	(Pd) Local alluvium overlying schist and granite	ML-CL, ML, CH, CL	Marginal – W, M, P	Poor – B, W	Marginal – M, P, W	Moderate	Low
10	Glenville	(Pd) Local alluvium overlying schist and granite	ML, ML-CL, SM	Fair – M, P, W	Fair – B, W	Marginal – M, P, W	Moderate	Moderate
11	Bermudian	(Tr) Alluvium on low terraces in floodplains	ML-CL, CL	Fair – P, K	Marginal – B, W	Fair – P, T, K	Moderate	Low
12	Rowland	(Tr) Alluvium on low terraces in floodplains	ML-CL, ML	Fair – P, W, K	Poor – B, W	Fair – P, T. W, K	Low	Low
13	Bowmansville	(Tr) Alluvium on low terraces in floodplains	ML-CL, CL, CH	Marginal – W, P, K	Poor – B, W	Marginal – W, P, K	Low	Low
14	Manassas	(Tr) Local alluvium overlying siltstone and sandstone	ML-CL, CL, ML, GC	Fair – P, W, K	Fair – B, W	Fair – P, T, W, K	Moderate	Moderate
15	Muck	(Cp) Organic sediments	OL, OH	Poor – W, O	Poor – B, W, O	Poor – W, O	Moderate	Low
18 19	Rocky Land and Very Rocky Land (Acid)	(Pd) Schist and granite	ML, SM	Marginal – D, R, M, P	Good	Poor – D, R, M, P	High	High
20	Meadowville	(Pd) Local alluvium overlying schist and granite	ML-CL, CL, ML, SM	Fair – M, P, W	Fair – B, W	Marginal – M, P, W	Moderate	Moderate
21	Manor	(Pd) Schist	ML, SM	Fair – M, P	Good	Poor – M, P	High	High

No.	Soil Name[1]	Physiographic Province/ Parent Material/ Landscape Position[2]	Typical USCS Classification[3]	Embankment Materials[4]	Embankment Foundation[4]	Core/Liner Materials[4]	Seepage Potential[5]	Erosion Potential[6]
23	Captina	(Pd) High terraces near streams	CL-ML, SM, SM-SC	Fair – P, W	Fair, B, W	Fair – P, T, W	Moderate	Moderate
24	Elioak	(Pd) Schist	ML-CL, MH, SM	Fair – M, P	Good	Fair – M, P	High	High
26	Bertie	(Cp) Silty Coastal Plain sediments	ML, CL	Fair – P, W	Fair – B, W	Marginal – P, W	Moderate	Moderate
27	Legore sil	(Tr) Diabase/diorite	ML, CL, MH-CH	Marginal – D	Good	Marginal – T, D	Low	Moderate
28	Montalto sil	(Tr) Diabase/diorite	ML, CL, MH-CH	Good	Good	Good	Low	Moderate
29	Legore st sil	(Tr) Diabase/diorite	ML, CL, MH-CH	Marginal – D	Good	Marginal – T, D	Low	Moderate
30	Huntington	(Pd, Cp) Aluvium on low terraces in Potomac River floodplain	ML-CL, CL, ML	Fair – P	Fair – B, W	Fair – P	Moderate	Low
31	Lindside	(Pd, Cp) Aluvium on low terraces in Potomac River floodplain	ML-CL, CL, ML	Fair – W, P	Marginal – B, W	Fair – W, P	Moderate	Low
32	Fairfax sil	(Pd) Silty upland terraces overlying schist and granite	ML, ML-CL, SM	Fair – P	Good	Marginal – P, M	Moderate	High
33	Melvin	(Pd, Cp) Alluvium on low terraces in Potomac River floodplain	ML-CL, CL, ML	Marginal – W, P	Poor – B, W	Marginal – W, P	Moderate	Low
34	Woodstown	(Cp) Sandy Coastal Plain sediments	SM-SC, SM, SC	Fair – P, W	Fair – W	Marginal – T, P, W	High	Low
35	Manteo	(Pd) Schist	CL, ML, SM	Marginal – D, M, P	Good	Poor – D, M, P	High	High
37 38	Beltsville sil Beltsville 1	(Cp) Silty uplands overlying dense gravelly Coastal Plain sediments or weathered schist and granite	ML, CL, ML-CL, SC	Fair – P, W	Good	Marginal – T, P, W	Moderate	Moderate
39	Othello	(Cp) Silty and clayey Coastal Plain sediments	ML-CL, ML, MH, CH, SM	Marginal – W, P	Poor – B, W	Marginal – W, P	Moderate	Low
40	Mecklenburg	(Tr) Diabase	ML-CL, MH, SM-SC	Fair – C	Marginal – Z	Fair – C	Low	Moderate

No.	Soil Name[1]	Physiographic Province/ Parent Material/ Landscape Position[2]	Typical USCS Classification[3]	Embank-ment Materials[4]	Embankment Foundation[4]	Core/Liner Materials[4]	Seepage Potential[5]	Erosion Potential[6]
41 42	Rocky Land and Very Rocky Land (Iredell Group)	(Tr) Diabase	ML-CL, CH, SC, SM	Marginal – R, D, C	Marginal – Z	Marginal – R, D, C	Low	Moderate
43	Masada gravelly loam	(Pd) Gravelly high terraces near streams	GM, ML, GC, CL	Good	Good	Fair – T	Moderate	Moderate
44	Caroline	(Cp) Silty and Clayey Coastal Plain sediments	ML, MH, CH	Fair – C	Marginal – B, C	Fair – C	Moderate	Moderate
45	Matapeake	(Cp) Silty Coastal Plain sedi-ments	ML-CL, CL, ML, SM	Fair – P	Good	Fair – P	Low	Moderate
46	Mattapex	(Cp) Silty Coastal Plain sedi-ments	ML-CL, ML, CL, SM	Fair – P, W	Good	Fair – P, W	Low	Moderate
47	Dragston	(Cp) Sandy Coastal Plain se-diments	SC, SM	Fair – W, P	Fair – B, W	Marginal – T, W, P	High	Low
48	Iredell	(Tr) Diabase	ML-CL, CH, SC	Fair – C, W	Marginal – Z	Fair – C, W	Low	Moderate
49	Lunt fine sandy loam	(Cp) Sandy to clayey Coastal Plain sediments	SM-SC, CH, SC	Fair – C, U	Marginal – B, C, U	Fair – T	High	Moderate
50	Iredell – Meck-lenburg st sil	(Tr) Diabase	ML-CL, MH, CH, SC	Fair – C, W, R	Marginal – Z	Fair – C, W, R	Moderate	Moderate
51	Keyport	(Cp) Silty and clayey Coastal Plain sediments	ML, CL, MH, CH	Fair – W	Fair – B, W	Fair – W	Low	Moderate
52	Elbert (Iredell Group)	(Tr) Local alluvium overlying diabase bedrock	CL, CH, MH-CH, SM-SC	Marginal – W, C	Poor – B, W, C	Marginal – W, C	Low	Low
53	Lenoir	(Cp) Silty and clayey Coastal Plain sediments	ML, ML-CL, MH-CH, CL	Fair – W	Marginal – B, W	Fair – W	Low	Moderate
54	Sassafras	(Cp) Sandy Coastal Plain se-diments	SM, SC	Fair – P	Good	Marginal – T, P	High	Moderate
55	Glenelg	(Pd) Schist	ML, SM	Fair – M, P	Good	Poor – M, P	High	High
56	Kempsville	(Cp) Silty and sandy Coastal Plain sediments	ML, SM, SM-SC, CL-ML, SC	Fair – P	Good	Marginal – T, P	Moderate	Moderate
57	Brecknock 1	(Tr) Baked sandstone (horn-fels)	ML-CL, CL	Fair – K	Good	Fair – K	Moderate	Moderate

No.	Soil Name[1]	Physiographic Province/ Parent Material/ Landscape Position[2]	Typical USCS Classification[3]	Embankment Materials[4]	Embankment Foundation[4]	Core/Liner Materials[4]	Seepage Potential[5]	Erosion Potential[6]
59	Orange	(Pd) Greenstone (metabasalt)	ML, CL, CH	Fair – C, W	Marginal – Z	Fair – C, W	Low	Moderate
60	Appling	(Pd) Granite and gneiss	ML, MH-CH, MH, SC	Good	Good	Fair – T	Moderate	High
61	Loamy/Gravelly Sediments	(Cp) Sandy and gravelly Coastal Plain sediments	CL, ML, MH, SM, GM, GC	Marginal – T, C, U	Marginal – B, C, U	Marginal – T, C	High	High
62	Brecknock gravelly silt loam	(Tr) Baked siltsone (hornfels)	ML-CL, ML	Fair – K	Good	Fair – K	Moderate	Moderate
63	Louisburg	(Pd) Granite and gneiss	SM	Good	Good	Marginal – T	Moderate	High
64	Silty/Clayey Sediments	(Cp) Silty and clayey Cretaceous-age Coastal Plain sediments	CH, MH, SC, CL, ML	Marginal – C, U	Poor – B, C, U	Marginal – C, T	High	High
65	Colfax	(Pd) Granite and gneiss	ML, CL, SC	Fair – W	Marginal – B,W	Fair – W, T	Low	Moderate
66	Lloyd	(Pd) Greenstone and schist	ML, MH	Good	Good	Good	Low	Moderate
67	Penn fsl	(Tr) Sandstone	SM, ML-CL, CL, ML	Fair – P, K, D	Good	Fair – P, K, D	High	High
68	Roanoke	(Pd) Clayey alluvium on low terraces in floodplains	CH, MH, CL, CL-ML, GM-GC	Marginal – W	Poor – B, W	Marginal – W	Low	Low
69	Enon	(Pd) Greenstone and schist	ML, MH-CH, ML-CL	Good	Fair – B	Good	Low	Severe
70	State	(Cp) Sandy alluvium on low terraces in floodplains	SM, SC, CL	Fair – P	Good	Marginal – T, P	High	Low
71	Bucks sil	(Tr) Siltstone	ML-CL, MH-CH, ML	Fair – P, K	Good	Fair – P, K	Moderate	Moderate
72	Bucks 1	(Tr) Sandstone	ML, CL, ML-CL	Fair – P, K	Good	Fair – P, K	Moderate	Moderate
73	Penn sil	(Tr) Siltstone and sandstone	ML-CL, ML, GC	Fair – P, K, D	Good	Fair – T, P, K, D	Moderate	High
75	Penn 1	(Tr) Sandstone and siltstone	ML-CL, ML, CL	Fair – D, P, K	Good	Fair – D, P, K	Moderate	High
76	Calverton 1	(Tr) Siltstone and sandstone	ML-CL, CL, MH-CH, SM-SC	Fair – W, K	Marginal – B, W	Fair – W, K	Low	Moderate
77	Penn sh sil	(Tr) Siltstone and sandstone	ML-CL, ML, GM-GC	Marginal – P, K, D	Good	Marginal – D, T, P, K	Moderate	High

No.	Soil Name[1]	Physiographic Province/ Parent Material/ Landscape Position[2]	Typical USCS Classification[3]	Embank-ment Materials[4]	Embankment Foundation[4]	Core/Liner Materials[4]	Seepage Potential[5]	Erosion Potential[6]
78	Calverton sil	(Tr) Siltstone and sandstone	ML-CL, ML, MH-CH, SM-SC	Fair – W, K	Marginal – B, W	Fair – W, K	Low	Moderate
79	Kelly	(Tr) Diabase and siltstone (hornfels)	ML-CL, CH, MH	Fair – K, C	Marginal – Z	Fair – K, C	Moderate	Moderate
80	Croton	(Tr) Siltstone and sandstone	ML-CL, ML, CH, MH, GM-GC	Marginal – W, K	Marginal – B, W	Marginal – W, K	Low	Low
83	Galestown	(Cp) Sandy Coastal Plain sediments	SM, SC	Fair – P	Good	Poor – T	High	Low
84	Fallsington	(Cp) Sandy Coastal Plain sediments	SM-SC, SM, SC	Marginal – W, P	Poor – B, W	Marginal – W, T	High	Low
85	Elkton	(Cp) Clayey Coastal Plain sediments	ML-CL, ML, CL, CH, MH	Marginal – W, C	Poor – B, W, C	Marginal – W, C	Low	Low
86	Klej	(Cp) Sandy Coastal Plain sediments	SM, SC	Fair – W	Fair – B, W	Poor – T	High	Low
87	Wickham	(Pd) Silty high terraces along streams	ML, SC, CL	Good	Good	Good	Low	Moderate
88	Hiwassee sil	(Cp) Silty high terraces along streams	ML, CL, MH	Good	Good	Good	Low	Moderate
89	Tidal Marsh	(Cp) Organic soils in recent alluvium along the tidal Potomac River	OL, OH	Poor – W, O	Poor – B, W, O	Poor – W, O	Moderate	Low
90	Augusta vfsl	(Pd, Cp) Silty and clayey alluvium on low terraces in floodplains	ML, CL, MH-CH, GC	Fair – W	Fair – B, W	Marginal – T, W	Low	Moderate
91	Birdsboro	(Tr) Silty and clayey alluvium on low to high terraces near streams	ML-CL, CL	Fair – P, W	Marginal – B, W	Fair – P, W	Low	Moderate
92	Raritan	(Tr) Silty and clayey alluvium on low to high terraces near streams	ML-CL, CH-MH, GM-GC	Fair – W, P	Marginal – B, W	Fair – W, P	Low	Moderate
104	Catlett	(Tr) Baked siltstone and sandstone (hornfels)	ML-CL, ML	Marginal – D, P, K	Good	Marginal – D, P, K	Moderate	Moderate

No.	Soil Name[1]	Physiographic Province/ Parent Material/ Landscape Position[2]	Typical USCS Classification[3]	Embank-ment Materials[4]	Embankment Foundation[4]	Core/Liner Materials[4]	Seepage Potential[5]	Erosion Potential[6]
110	Augusta 1	(Pd, Cp) Silty and clayey alluvium on low terraces in floodplains	ML, CL, MH-CH, GC	Fair – W	Fair – B, W	Marginal – T, W	Low	Moderate
112	Augusta sl	(Pd, Cp) Silty and calyey alluvium on low terraces in floodplains	ML, CL, MH-CH, GC	Fair – W	Fair – B, W	Marginal – T, W	Low	Moderate
113	Fairfax gr sil	(Pd) Silty and gravelly upland terraces overlying schist and granite	ML, ML-CL, SM, GM	Fair – P	Good	Marginal – P, T, M	High	High
114	Masada fsl	(Pd) Gravelly high terraces along streams	GM, ML, GC, CL	Good	Good	Fair – T	Moderate	Moderate
115	Hiwassee fsl	(Pd) Silty high terraces along streams	ML, CL, MH	Good	Good	Good	Low	Moderate
116	Christiana	(Cp) Silty and clayey Cretaceous-age Coastal Plain sediments	MH, CH	Poor – C, U	Poor – U, C, B	Marginal – C	Moderate	Moderate
117	Marsh (Fresh)	(Cp) Organic soils and alluvium along streams	OL, OH	Poor - W, O	Poor – B, W, O	Poor – W, O	Moderate	Low
118	Marine Clay	(Cp) Clayey and silty Cretaceous-age Coastal Plain sediments	CH, MH	Poor – C, U	Poor, U, C, B	Marginal – C	Moderate	High
120	Altavista	(Cp) Sandy and clayey alluvium on low terraces in floodplains	CL, CL-ML, SC, SM-SC	Fair – P, W	Fair - W	Fair – P, W	Moderate	Moderate
128	Montalto st sil	(Tr) Diabase/diorite	ML, CL, MH-CH	Fair – R	Good	Fair – T, R	Low	Moderate
129	Montalto r sil	(Tr) Diabase/diorite	ML, CL, MH-CH	Fair – R	Good	Fair – T, R	Low	Moderate
132	Mayodan	(Tr) Sandstone conglomerate	SM, ML, SM-SC, MH	Good	Good	Good	Low	Moderate
141 142	Rocky Land and Very Rocky Land (Orange Group)	(Pd) Greenstone (metabasalt)	ML, ML-CL, CH	Marginal – R, D, C	Marginal – Z	Marginal – R, D, C	Low	Moderate

No.	Soil Name[1]	Physiographic Province/ Parent Material/ Landscape Position[2]	Typical USCS Classification[3]	Embank-ment Materials[4]	Embankment Foundation[4]	Core/Liner Materials[4]	Seepage Potential[5]	Erosion Potential[6]
148	Iredell – Meck-lenburg sil	(Tr) Diabase	ML-CL, MH, CH, SC	Fair – C, W	Marginal – Z	Fair – C, W	Low	Moderate
149	Lunt sil	(Cp) Clayey and sandy Coast-al Plain sediments (includes Cretaceous-age sediments)	SM-SC, CH, MH	Marginal – C, U	Marginal – U, B, C	Marginal – C	Moderate	Moderate
152	Elbert (Orange Group)	(Pd) Local alluvium overlying Greenstone (metabasalt)	CL, CH, MH-CH	Marginal – W, C	Poor – B, W, C	Marginal – W, C	Low	Low
232	Fairfax 1	(Pd) Clayey and silty upland terraces overlying weathered schist and granite	ML, MH-CH, MH, ML-CL	Fair – P	Good	Fair – P	Moderate	High
273	Readington	(Tr) Siltsone and sandstone	ML-CL, CL, ML	Fair – P, K, D, W	Good	Fair – P, K, D, W	Moderate	Moderate

NOTES:

Soil Name[1] (56-96-PFM)

Soil names are taken from the **Soil Survey of Fairfax County, Virginia, Series 1955, No. 11, Issued May 1963**. Additional soil series, not included in the original survey, occur in revised soil maps of Fairfax County. Since the original soil survey in 1955, the USDA Soil Conservation Service has continued to revise and update its list of soils found state-wide in Virginia. Property descriptions and interpretations for some soils were modified as more information was gathered, and some soil names were changed. As a result, some soil series used in Fairfax County may not coincide in properties and interpretations with the same soil names used elsewhere in Virginia. Properties and engineering interpretations in this table are specific to Fairfax County, and are based on surveys and data gathered by the County since the original survey.

Soil names include modifiers that indicate surface texture (proportion of sand, silt, clay, gravel, stones, etc.). Differences in surface texture often indicate parent material differences and reflect other differences in the soil which may affect engineering properties. The following abbreviations (USDA texture name) are used in this table: fsl (fine sandy loam), gr (gravelly), l (loam), r (rocky), sh (shaly), sil (silt loam), sl (sandy loam), st (stony), vfsl (very fine sandy loam).

Physiographic Province/ Parent Material/ Landscape Position[2] (56-96-PFM)

Physiographic Province, Parent Material, and Landscape Position defines general geologic area, source of soil constituent, and/or landscape setting. Physiographic Province is defined as: Tr = Triassic, Pd = Piedmont, and Cp = Coastal Plain. Detailed geologic maps are available from the U.S. Geological Survey.

Typical USCS Classification[3] (56-96-PFM)

Typical Unified Soil Classification System (USCS) Classifications listed here are estimates based on limited laboratory analyses (published data include the Soil Survey of Fairfax County, Virginia and F.H.A. Report No. 373 "Engineering Soil Classification For Residential Development") and on observations and test data assembled by the County. Classes typically found in each soil type are listed. Site-specific variations occur within soil types. These soil classifications should be used for planning purposes only and should not replace on-site investigations for significant dam structures.

Key to General Ratings For Embankment Materials, Embankment Foundation, and Core/Liner Materials[4]

The design of an earthen structure should be preceded by careful investigation of both the cut and fill areas. Soils typically occur as horizons or layers that change significantly in gradation and other physical properties with depth and horizontal distance. For example, the Iredell (48) series consists of less than 1' (0.3m) of silts overlying 1' to 3' (0.3m to 1m) of highly plastic clay, which in turn overlies sandy to clayey decomposed bedrock of variable depth. The depth to bedrock or large boulders in the Iredell soils may vary from 3' to 15' (1m to 4.6m). For these and other soils, care should be taken in engineering investigations to identify significant soil strata changes that occur over short distances. Previous excavation or filling activities may significantly alter site conditions.

As a general rule in embankment construction, all visible organic debris such as roots and limbs should be removed from the fill material prior to compaction to a specified density. Soils with organic matter content exceeding five percent by weight should not be used as structural fill. Stones greater than 4" to 6" (100mm to 150mm) in diameter should be removed from the fill material. It is essential that a good bond be established between the soils in the dam and in the foundation by removing loose organic debris, organic-rich soils, and soft soils prior to compacting and scarifying the subgrade.

For reestablishment of vegetation after construction, a minimum of 6" (150mm) of topsoil, limed and fertilized, should be placed on the embankment surface.

Ratings for **Embankment Materials** evaluate the soil as a source of fill for embankment construction. Ratings apply to the upper 5' (1.5m) of in-situ soil material and consider that mixing of the soil materials will occur during construction operations.

Ratings for **Core/Liner Materials** evaluate the soil as a source of low-permeability materials to be used as an impervious soil core within the dam or as an upstream liner above highly permeable substrata to minimize seepage loss. Segregation of acceptable soil strata from surrounding soils is usually necessary to minimize contamination.

Ratings for **Embankment Foundations** are based on the ability of the natural (undisturbed) soil to support an embankment without excessive settlement occurring.

Ratings:

Good	=	No significant problems in natural undisturbed soils.
Fair	=	Minor potential problems affecting design or construction.
Marginal	=	Significant problems that must be considered in design and construction.

Poor = Major problems that must be addressed during the design and construction to ensure satisfactory performance of structures.

Key to Problems and Characteristics For Embankment Materials, Embankment Foundation, and Core/Liner Materials

B = Low bearing values due to soft or saturated soil strata may provide marginal to poor support for the dam and result in significant total or differential settlement.

C = High shrink-swell clays are difficult to work or compact under certain moisture contents (too wet or too dry). These clays are typically suitable for liner materials, but may be difficult to compact properly.

D = Shallow depth to bedrock results in a thin soil layer and lack of sufficient materials for the embankment or core. Suitable soil material may need to be imported from off-site.

K = The bedrock disintegrates (slakes) rapidly when exposed to surface or subsurface weathering, which may lead to embankment instability unless proper gradation is attained during compaction.

M = High mica content makes the soil difficult to compact and increases the susceptibility to piping and embankment slope failure.

O = High organic matter content (organic strata, loose debris, or organic enrichment in mineral horizons) results in compression and differential settlement under the embankment foundation. The organic materials and organic-enriched soils (greater than 5 percent organic matter) are difficult to compact properly and will decay over time, reducing the embankment and core stability.

P = Piping hazard (internal erosion and channeling) may occur in the dam foundation as a result of no or inadequate core construction, and within embankments because of poor compaction.

R = High content of rocks or stones in the soil interferes with compaction, grading, workability.

T = Medium to coarse textures (SM or coarser) are suitable for the shell but not the core of the dam.

U = Potentially unstable slopes resulting in slope failure or slope creep may destabilize the dam. Slope failures may occur unless the embankments are constructed at slopes of 4H:1V or flatter.

W = High seasonal water tables result in wet conditions during certain periods of the year, adversely affecting workability and compaction. Wetness problems are minimized during dry periods of the year.

Z = Embankment foundation support is poor in the plastic clay layer, good in underlying saprolite or bedrock.

Seepage Potential[5]

Seepage potential is based on permeability of the near-surface soils and depth to permeable saprolite, fractures bedrock, or other permeable strata. These properties are evaluated based on the potential for seepage loss from the excavated areas within the reservoir, emergency spillway and under the embankment.

Soils with a **high seepage potential** have moderately rapid or rapid permeability in the near-surface soils or have highly permeable saprolite, fractured bedrock, or other permeable strata. Soils with a **moderate seepage potential** have a moderate permeability or have permeable saprolite, bedrock, or other strata, often deeper than 4' (1.2m). In some predominantly silty or clayey Coastal Plain soils, lateral seepage may occur within permeable strata. Moderately slow to slowly permeable soils which are not likely to be underlain by permeable saprolite, bedrock, or other strata have a **low seepage potential**.

Erosion Potential[6]

Erosion potential is based on the Universal Soil Loss Equation adapted for soils under construction site conditions. Soil erodibility is affected by texture (relative proportion of sand, silt, and clay), rock content, permeability, structure, and slope (natural or man-made).
Soils with a **low erosion potential** are not highly erodible, rarely exceeding soil loss tolerances except on steep unprotected cuts.
Soils with a **moderate erosion potential** are moderately erodible on B (2-7%) slopes and highly erodible on C (7-14%) slopes or greater (exceeding the soil loss tolerance). Sheet, rill and shallow gully erosion can be expected on unprotected soils during a severe storm.
Soils with a **high erosion potential** are highly erodible, exceeding soil loss tolerances even on B (2-7%) slopes. Sheet and rill erosion, with the formation of numerous gullies can be expected on unprotected soils in a severe storm.

6-1606 Maintenance and Safety Design Requirements

6-1606.1 Safety Considerations. The following design considerations address safety concerns related to dams and impoundments. Principal safety concerns involve child access to various components of the stormwater management facility; therefore, design features should discourage child access. Safety features should also be consistent with the requirements of § 6-0303.

6-1606.1A Trash racks and other debris control structures shall be sized to prevent entry by children. Bar spacing on any debris control structure shall be no greater than 12" (300mm) in any direction, with the preferred spacing being 6" (150mm).

6-1606.1B Fencing or other barriers shall be required around spillway structures having open or accessible drops in excess of 3' (900mm).

6-1606.1C Embankment and pond slopes generally should be no steeper than 3H:1V. For dam embankments exceeding 15' (4.5m) in height, a 6' to 10' (1.8m to 3m) wide bench should be provided at intervals of 10' to 15' (3m to 4.5m) in height, particularly if slopes are steeper than 3H:1V. Slopes steeper than 2.5H:1V shall not be permitted without approval by the Director.

6-1606.1D Shorelines along wet ponds in areas accessible to the public should incorporate a shallow bench 1' to 2' (0.3m to 0.6m) deep extending 5' to 10' (1.5m to 3m) from the shoreline.

6-1606.1E Safety signs shall be placed in areas near wet ponds and spillway structures.

6-1606.2 Maintenance Considerations. All dams shall be designed with the maintenance design considerations stated in § 6-1306. In addition, the following maintenance provisions shall also be considered in designing the dam.

6-1606.2A Sediment forebays should be considered at most stormwater management facilities having permanent pools (i.e., retention ponds). Sediment forebays should be located near major pond inflow locations and should have sufficient storage and depth to trap projected sediment over a 10- to 20-yr period. On-site sediment disposal areas (decanting basins) should be considered near wet ponds to reduce or eliminate hauling and dumping costs to offsite disposal areas. Maintenance access shall be provided, including access to the sediment forebay. The maintenance access shall be stabilized to provide passage of heavy equipment.

6-1606.2B Low level drains, generally sluice gates, should be designed with rising stems, particularly when the stem is located in the reservoir area. Non-rising stems are not acceptable except in instances where the stem is easily accessible for frequent maintenance (cleaning and greasing).

6-1606.2C Internal drainage systems in dam embankments (e.g., drainage blankets, toe drains) should be designed such that the collection conduits (e.g., perforated PVC pipe) discharge downstream of the dam at a location where access for observation is possible by maintenance personnel.

6-1606.2D Adequate erosion protection is required along the contact between the face of the embankment and the abutments. Runoff from rainfall concentrates in these areas and can reach erosive velocities depending on the gutter slope and dam height. Although a sod gutter will be satisfactory for most small dams, an evaluation should be made at each dam to determine if some type of gutter protection other than sod is required. For most dams, a riprap gutter is preferred rather than a paved concrete gutter.

6-1606.2E For permanent impoundments (wet ponds), the upstream face of a dam may be protected against wave erosion by placement of a layer of riprap over a layer of filter material. Riprap not smaller than VDOT Standard Class II is required for this purpose. Vegetative protection will usually be sufficient on the upstream face of smaller impoundments if the effective reservoir fetch is less than 500' (152m) and the dam embankment soils are not highly erosive.

6-1606.2F Trees, shrubs, or any other woody plants shall not be planted on the dam embankment or adjacent areas extending at least 10' (3m) beyond the embankment toe and abutment contacts.

6-1606.2G (76-02-PFM) Access shall be provided to all areas of an impoundment requiring observation or regular maintenance. These areas include the dam embankment, emergency spillway, lake shoreline, principal spillway outlet, stilling basin, toe drain, riser structure, extended drawdown device, and likely sediment accumulation areas. The schematic pond layouts shown in Plates 49-6 (49M-6), 50-6 (50M-6), 56-6 (56M-6) and 57-6 (57M-6) show typical maintenance access road locations. An access road detail is provided in Plate 70-6 (70M-6). A 20' (6.1m) cleared access easement shall be provided from the access entrance along the downstream side of the embankment toe to the outlet channel. Unauthorized vehicular access shall be controlled with a standard access road gate (see Plates 71-6 (71M-6) and 71A-6 (71AM-6). The Director may allow the use of a cable barricade (see Plate 10-8 (10M-8)) in lieu of a standards access road gate for ponds that control a watershed of less than 100 acres (40ha) and where the cable barricade would be more compatible with the proposed development and would be sufficient to restrict unauthorized vehicular access to the pond.

6-1607 Minimum Required Construction Standards, Specifications and Inspection Requirements

6-1607.1 Construction Specifications for Category "D" Dams (see Plate 64-6 (64M-6). The minimum required construction specifications outlined below are intended for Category D dams, except where specific reference is made to Category A, B, and C dams. These specifications should be considered as minimum requirements with the understanding that more stringent specifications may be required dependent on individual site conditions as evaluated by the project geotechnical engineer and/or DPWES. Therefore, in all cases final construction specifications tailored to each individual project shall be included on the construction plans submitted to DPWES. In general, any construction items not addressed in the final dam construction specifications should adhere to VDOT standards and specifications. In all cases, the final dam construction specifications shall take precedence over VDOT specifications.

6-1607.1A Foundation and Abutment Preparation:

6-1607.1A(1) Extent. The foundation and abutment area is defined to extend to a distance of 10' (3m) beyond all limits of the planned facilities.

6-1607.1A(2) Clearing, Grubbing and Stripping. The foundation and abutment area shall be cleared, grubbed and stripped of all vegetation, topsoil and/or organic soil and “any other unsuitable materials,” as specified by the construction plans. “Other unsuitable materials” should be defined in the specification by the designer along with the estimated minimum depth of undercut where possible.

6-1607.1A(3) Control of Surface and Groundwater. The landowner and his contractor shall be responsible for removal and control of any surface water and groundwater which would adversely affect construction.

6-1607.1A(4) Subgrade Preparation and Approval. After clearing, grubbing, stripping and removing any other unsuitable materials, the subgrade shall be proofrolled with compaction equipment under the observation of a qualified engineer. The compaction equipment should be the heaviest possible equipment that will not cause disturbance of suitable subgrade soils. If, in the opinion of the Director or the engineer, excessively soft or unsuitable materials are disclosed, these unsuitable materials shall be removed and replaced with compacted fill. Where rock is exposed after stripping or undercutting, all loose rock material shall be removed prior to placing compacted fill. Rock subgrades shall be inspected and approved by the inspector or engineer prior to placement of fill.

6-1607.1B Conduits/Structures:

6-1607.1B(1) Subgrade. After excavation for the planned conduit/structure, the subgrade should be inspected and approved by the engineer. Any soft or unsuitable soils shall be removed and replaced with compacted fill meeting the requirements in § 6-1607.1C.

6-1607.1B(2) Pipe Bedding (57-96-PFM). After approval of the subgrade, a concrete cradle shall be provided below the pipe for the distance specified on the construction plans (see Plate 64-6 (64M-6)) for minimum requirements). The remaining downstream one-third of the pipe shall be bedded in accordance with Plate 18-10 (18M-10) or 19-10 (19M-10). The cradle and bedding should be placed to the spring line of the conduit.

6-1607.1B(3) Backfill. Backfill shall meet the same requirements set forth in § 6-1607.1C.

6-1607.1B(4) Pipe. Principal spillway pipe shall be reinforced concrete pipe which meets the following specifications:

6-1607.1B(4)(a) (68-00-PFM). For dams having permanent pools (wet ponds), reinforced concrete pressure pipe shall be used which meets AWWA specifications C300, C301, C302 or ASTM specification C361.

6-1607.1B(4)(b) For dams associated with dry or extended dry (BMP) detention facilities, reinforced concrete low head pressure pipe shall be used which meets ASTM specification C361.

6-1607.1B(4)(c) Reinforced concrete pipe strength shall be in accordance with VDOT Standard PC-1, with Class III being the minimum strength permitted.

6-1607.1B(5) Trash Racks. Trash rack members shall be A36 steel. When reinforcing bars are used as cage members, they shall be grade 60. All components shall be galvanized in accordance with VDOT Specification 241. Trash racks shall be attached to a concrete spillway structure with stainless steel anchor bolts.

6-1607.1C Earthwork:

6-1607.1C(1) Compacted Fill. Compacted fill shall not be placed prior to performing the required foundation and abutment preparation, or on any frozen surface. Compacted fill shall extend to the fill limit lines and grades indicated by the approved construction plan. Compacted fill material shall be of the type classification symbol specified by the designer, or of a better quality material as defined by the Unified Soil Classification System. Also, restrictions on the liquid limits and plasticity index of the material may be included where applicable. Compacted fill shall consist of material free of organic matter, rubbish, frozen soil, snow, ice, particles with sizes larger than 3" (75mm) or other deleterious material.

6-1607.1C(2) Compacted fill shall be placed in horizontal layers of 8" to 12" (200mm to 300mm) in loose thickness. Actual lift thickness will be specified on a case-by-case basis. The moisture content shall be controlled such that compaction is achieved without yielding of the surface. Each layer shall be uniformly compacted with suitable compaction equipment to at least 95% of Standard Proctor Maximum Density in accordance with ASTM D-698, AASHTO T-99, or VDOT specifications. Any layer of fine-grained fill which becomes smooth under compaction or construction traffic should be scarified to a depth of 2" (50mm) to allow adequate bonding between layers.

6-1607.1C(3) Compacted fill with a moisture content which will not permit compaction to the specified density standard shall be scarified dried or wetted as necessary to permit proper compaction.

6-1607.1C(4) Riprap. Rock fill at the embankment surface for the purpose of protecting the embankment against weathering, wave action, etc., shall consist of Class II riprap in accordance with VDOT specifications.

6-1607.1D Field Density Testing:

6-1607.1D(1) Tests of the degree (%) of compaction of the compacted fill shall be performed as part of the permittee's normal quality control program for construction of the dam. Tests will be made concurrently with the installation of the compacted fill and the contractor shall coordinate his work so that the testing can be accomplished. Should the results of the tests indicate that the specified degree of compaction is not obtained, the fill represented by such tests shall be reworked, sprinkled with water or scarified, dried as required and retested until the specified minimum degree of compaction is achieved.

6-1607.2 Construction Inspection Requirements:

6-1607.2A Category A, B, or C Dams (defined in Plate 64-6 (64M-6)). A qualified engineer as described in § 6-1605.2B of these design standards must observe foundation and abutment preparation, installation of cutoff trench, internal drainage system, outlet pipe or culverts, riser structure foundations, fill placement, and any other geotechnical-related items. The frequency of the observations and testing shall be adequate for the geotechnical engineer to state, in his professional opinion, that the specific items observed and tested were installed in accordance with the approved construction plans and specifications. Compaction in the field shall be monitored based on laboratory density test results. At least 1 field density test shall be conducted per 10,000 ft^2 (930 m^2) of compacted area per lift with at least 1 test occurring every other lift. Additional tests must be performed if there is any change in material. The location of tests are to be representative of the area under construction. Within 30 days following the completion of construction of the dam, an inspection report shall be provided to DPWES for review. As a minimum, these reports should state subgrade material and condition, grouting, fill material classification, in-place density and moisture content, location and elevation of tests performed, interior drainage material, filter type, location and conduit/structure grades.

6-1607.2B Category D Dams. The permittee shall be responsible for providing all quality control procedures necessary to ensure conformance with the approved plans and specifications. Within 30 days following the completion of construction of the dam, an inspection report shall be provided to DPWES for review. This report shall include all test results set forth in § 6-1607.2A.

6-1607.2C If there is any question as to the physical integrity of a constructed dam or stormwater management facility because of inadequate construction documentation, inadequate inspection reporting, or some other apparent inadequate condition, the Director may require a geotechnical engineering study after construction to verify that the facility has been constructed in accordance with § 6-0000 et. seq.

6-1607.3 As-Built Requirements and Certification:

6-1607.3A (57-96-PFM) Upon satisfactory completion, inspection, and approval of all components of the facility, as-built plans shall be prepared in accordance with the Zoning Ordinance, § 17-300, and the Subdivision Ordinance, § 101-2-5 of the Code.

6-1607.3B All existing plans to be modified for use as the as-built plan shall be redrafted where necessary so that the information is accurate and readable. Information included on the as-built plan shall include, at a minimum, the following information:

6-1607.3B(1) A profile (with spot elevations) of the top of dam

6-1607.3B(2) A cross-section (with spot elevations) of the emergency spillway at the control section

6-1607.3B(3) A profile (with spot elevations) along the centerline of the emergency spillway

6-1607.3B(4) A profile along the centerline of the principal spillway extending at least 100' (30m) downstream of the toe of the embankment

6-1607.3B(5) All tops, throats and invert elevations

6-1607.3B(6) All pipe, orifice and weir sizes and invert elevations

6-1607.3B(7) The elevation of the principal spillway crest

6-1607.3B(8) The elevation of the principal spillway conduit invert (inlet and outlet)

6-1607.3B(9) The elevation of the emergency spillway crest

6-1607.3B(10) Spot elevations around the entire pond/dam adequate to depict the shape and size

6-1607.3B(11) Spot elevations along the top and crest of the dam width

6-1607.3B(12) Spot elevations through the drainage way to the riser structure

6-1607.3B(13) Notes and measurements to show that any special design features were met

6-1607.3B(14) Statement regarding seeding and fencing

6-1607.3B(15) Show all drainage and access easements for maintenance of the pond/dam and related facilities with Deed Book and Page Number

6-1607.3C Each as-built plan shall have a Engineer's or Surveyor's certification statement and seal. Except for Category D dams, the certification of all geotechnical work will be by the geotechnical engineer of record. The certification shall state as follows:

6-1607.3C(1) In accordance with the Zoning Ordinance, § 17-300 and § 18-802, and the Subdivision Ordinance, § 101-2-5 of the Code, I, (submitting engineer's name), do hereby certify that this as-built conforms to the approved plans, except as shown, which represents actual conditions on this site as of this date.

(submitting engineer's signature/date) (seal)

6-1607.3C(2) I have reviewed the as-built plan and hereby certify that the geotechnical aspects of the embankment dam/pond were constructed in accordance with the approved plans, except as indicated below, which represents the actual conditions of the dam on this site as of this date.

(geotechnical engineer's signature/date) (seal)

6-1607.3C(3) All storm/sanitary structures fall within their respective easements and all dedications and all off-site easements are recorded in DB ___, at PG ___.

6-1608 Operation, Maintenance, and Inspection Guidelines

6-1608.1 A Private Maintenance Agreement with the Board of Supervisors shall be executed by the owner for all privately maintained stormwater management facilities prior to construction plan approval in accordance with § 6-0303.

6-1608.2 The following are general operation, maintenance, and inspection standards for all stormwater management facilities (including dams) not maintained by the County. The guidelines presented here are not intended to be all inclusive and specific facilities may require special measures not discussed here.

6-1608.3 Operation and Maintenance:

6-1608.3A Embankment. The dam embankment should maintain a thick, healthy grass cover over the embankment which is free of trees and brush. This type of cover will assist in stabilizing the surfaces of the dam as well as increase the ease of inspecting the dam.

6-1608.3A(1) The embankment should be mowed periodically during the growing season with the last cutting occurring at the end of the growing season. The grass cover should not be cut to less than 6" to 8" (150mm to 200mm) in height.

6-1608.3A(2) If necessary, the embankment should be limed, fertilized and seeded in the fall after the growing season. The amount of lime and fertilizer should be based on soils test results. The type of seed shall be consistent with that originally specified on the construction plans.

6-1608.3A(3) All erosion gullies noted during the growing season should be backfilled with topsoil, reseeded and protected until revegetated.
6-1608.3A(4) All bare areas and pathways on the dam embankment should be properly seeded and protected to eliminate the potential for erosion.

6-1608.3A(5) All animal burrows should be backfilled and compacted. Measures should be taken to remove the animals from the area.

6-1608.3A(6) All trees, woody vegetation, and other deep-rooted growth, including stumps and associated root systems, are to be removed from the dam embankment and adjacent areas to at least 10' (3m) beyond the embankment toe and abutment contacts. The old root system should be removed and the excavated volume replaced and compacted with material in character with the surrounding area. All seedlings should be removed at the first opportunity. Similarly, any vine cover and brush should be removed from the dam embankment to allow for proper and complete dam inspection.

6-1608.3B Spillways:

6-1608.3B(1) Spillway structures shall be cleared of debris periodically, and after any significant rainfall event if inspection reveals significant blockage.

6-1608.3B(2) During low water conditions, concrete spillway structures such as outlet conduits, risers, weir structures, etc, shall be inspected to determine if water is passing through any joints or other structure contacts. The condition of any concrete structure should be checked for cracks, spalling, and broken or loose sections. Any cracked or spalled areas should be cleaned and refilled with an appropriate patching concrete. Any extensive leakage, spalling or fractures should be inspected by a Professional Engineer and his recommendations followed.

6-1608.3B(3) Stilling basins and discharge channels shall be cleared of brush at least once per year.

6-1608.3B(4)) Trash racks and locking mechanisms shall be inspected and tested periodically to make sure they are intact and operative.

6-1608.3B(5) Vegetated emergency spillway channels should be mowed at the time of embankment mowing. The grass in the emergency spillway should not be cut to less than 6" to 8" (150mm to 200mm) in height. The emergency spillway approach and discharge channels should be cleared of brush and trees periodically. After any flow has passed through the emergency spillway, the spillway crest (control section) and exit channel should be inspected for erosion. All erosion areas should be repaired and stabilized.

6-1608.3C Toe Drains, Low-Level Lakes, Drains and Sluice Gates:

6-1608.3C(1) All sluice gates (or other types of gates or valves used to drain an impoundment) should be operated periodically to insure proper functioning. At those times, the gate and stem should be lubricated and all exposed metal shall be painted to protect it from corrosion.

6-1608.3C(2) Toe drains or other internal drainage outlets should be cleared of debris, brush, and silt at least once per year to allow and ensure the free flow of water.

6-1608.3D Additional Maintenance Items:

6-1608.3D(1) The dam should be inspected periodically to ensure that motorcycle, ATV, and other vehicles are not operating on the dam embankment or emergency spillway.

6-1608.3D(2) The common areas and other access points to the dam should be inspected to ensure that plantings, fences, or other obstructions are not placed such that access to the dam is impeded.

6-1608.4 Inspection:

6-1608.4A At a minimum, an annual inspection (by a person with experience in dam inspection) should be performed following a mowing and brush removal operation. Important items to look for during an inspection include: any evidence of movement within the dam or at the abutments; seepage anywhere along the dam toe; excessive erosion or other damage to the embankment or emergency spillway; any growth of trees or underbrush in the dam embankment or in the emergency spillway; heavy pedestrian and/or vehicular traffic on the dam embankment or emergency spillway; animal burrows or wave action damage along the dam embankment; and, for any stormwater management facility where a portion of the upstream ponding area is left "natural" or in an undisturbed condition within a conservation easement, any disturbance within this area should be noted.

6-1608.4B The dam inspection checklist provided by the Virginia Division of Soil and Water Conservation in the publication entitled "Safety Evaluation of Small Dams" also should be used when making annual dam inspections.

6-1700 POLICY ON WHAT MAY BE DONE IN CHESAPEAKE BAY PRESERVATION AREAS (38-93-PFM) (79-03-PFM)

6-1701 General Information

6-1701.1 (94-06-PFM) Certain areas of the County have been designated Chesapeake Bay Preservation Areas (CBPAs) and divided into Resource Protection Areas (RPAs) and Resource Management Areas (RMAs) that are subject to the criteria and requirements contained in Chapter 118 (Chesapeake Bay Preservation Ordinance) of the Code. RPAs are protected from most development because, left intact, they function to improve and protect water quality. RMAs are regulated to protect RPAs and water resources from degradation resulting from development and land disturbing activity.

6-1701.2 A map of CBPAs has been adopted by the Board. Where RPA and RMA boundaries on the adopted map differ from boundaries as determined on a site-specific basis from the text of Chapter 118 (Chesapeake Bay Preservation Ordinance) of the Code, the text shall govern.

6-1701.3 The site-specific boundaries of the RPA shall be delineated on all preliminary plans, site plans, subdivision plans, grading plans, public improvement plans, record plats, and all other plans of development in accordance with Chapter 118 (Chesapeake Bay Preservation Ordinance) of the Code and subject to the approval of the Director. (79-03-PFM)

6-1701.4 (94-06-PFM, 79-03-PFM) Chapter 118 (Chesapeake Bay Preservation Ordinance) of the Code requires that a reliable, site-specific evaluation shall be conducted to determine whether water bodies on or adjacent to development sites have perennial flow and that RPA boundaries shall be adjusted, as deemed necessary by the Director, on the site, based on this evaluation of the site. The evaluations performed by the Department of Public Work and Environmental Services (DPWES) that are the basis for the perennial streams depicted on the adopted map of CBPAs satisfy this requirement. Water bodies identified as perennial on the adopted map of CBPAs are presumed to be perennial and may only be reclassified as intermittent based on additional studies performed in accordance with this Article and Chapter 118 (Chesapeake Bay Preservation Ordinance) of the Code.

6-1702 Use Regulations in Chesapeake Bay Preservation Areas (79-03-PFM, 86-04-PFM))

6-1702.1 Unless an exception is approved by the Exception Review Committee or the Board of Supervisors, as provided for in Chapter 118 (Chesapeake Bay Preservation Ordinance) of the Code, all newly proposed buildable subdivision lots in or adjacent to an RPA must contain sufficient area of land outside the RPA to allow development of the lot without encroachment upon the RPA. (79-03-PFM)

6-1702.2 Land development and redevelopment may be allowed within an RPA if otherwise permitted by the Zoning Ordinance and subject to the requirements of the PFM and to the performance criteria of Chapter 118 (Chesapeake Bay Preservation Ordinance) of the Code, if it is water-dependent development as defined as § 118-1-6 of Chapter 118 (Chesapeake Bay Preservation Ordinance) of the Code, is considered to be redevelopment, is exempted, or for which an exception allowing the land development is approved in accordance with Chapter 118 (Chesapeake Bay Preservation Ordinance) of the Code.

6-1702.3. No part of any building lot in a cluster subdivision may extend into a Resource Protection Area, except as provided in Part 6 of Article 9 of the Zoning Ordinance for cluster subdivisions in the R-C, R-E and R-1 Districts and for cluster subdivision in the R-3 and R-4 Districts which have a minimum district size of two (2) acres but less than three and one-half (3.5) acres, and § 101-2-8 of the Code for cluster subdivisions in the R-2 District and cluster subdivisions in the R-3 and R-4 Districts which have a minimum district size of three and one-half (3.5) acres or greater.

6-1702.4 All wetlands permits required by law shall be obtained prior to commencing land disturbing activities. No land disturbing activity shall commence until all such permits have been obtained by the applicant and evidence of such permits has been provided to the Director. For those activities regulated under general permits for which the issuing agencies do not normally provide written confirmation of permit issuance, a copy of the general permit(s) and a statement describing the proposed activity and certifying compliance with all applicable permit conditions will serve as the required evidence. Wetlands permits include both COE Permits and Virginia Water Protection Permits.

6-1703 Water Quality Impact Assessments

6-1703.1 A Water Quality Impact Assessment (WQIA) is required for any development or redevelopment within an RPA unless waived by the Director or exempted under Chapter 118 (Chesapeake Bay Preservation Ordinance) of the Code.

6-1703.2 The Director shall require a WQIA for development or redevelopment within an RMA if the Director determines that such an assessment is necessary because of the unique characteristics of the site or because the intensity of the proposed development may cause significant impacts on the adjacent RPA.

6-1703.3 WQIAs, as required, shall be submitted to the Director for review in conjunction with the submission of a plan of development. Unless modified by the Director, a WQIA shall be performed in accordance with Chapter 118 (Chesapeake Bay Preservation Ordinance) of the Code.

6-1704 GUIDELINES FOR DETERMINING LOCATIONS OF RESOURCE PROTECTION AREAS AND IDENTIFYING WATER BODIES WITH PERENNIAL FLOW. (79-03-PFM)

6-1704.1 (94-06-PFM) Resource Protection Area (RPA) boundary delineation studies and the identification of water bodies with perennial flow shall be performed by the methods described herein or other acceptable methods as determined by the Director.

6-1704.2 The RPA shall include any land characterized by one or more of the following features:

6-1704.2A A tidal wetland;

6-1704.2B A tidal shore;

6-1704.2C A water body with perennial flow;

6-1704.2D A nontidal wetland connected by surface flow and contiguous to a tidal wetland or water body with perennial flow;

6-1704.2E A buffer area as follows:

6-1704.2F (1) (94-06-PFM) Any land within a major floodplain ["Major floodplain" means those land areas in and adjacent to streams and watercourses subject to continuous or periodic inundation from flood events with a one (1) percent chance of occurrence in any given year (i.e., the 100-year flood frequency event) and having a drainage area equal to or greater than three hundred and sixty (360) acres (146 ha)[1].];

6-1704.2F(2) (94-06-PFM) Any land within 100 feet (30.5m) [1] of a feature listed in § 6-1704.2A through § 6-1704.2D. The full buffer area shall be designated as the landward component of the RPA not withstanding the presence of permitted uses, encroachments, and permitted vegetation clearing.

6-1704.3 Designation of the RPA components listed in § 6-1704.2A through § 6-1704.2D shall not be subject to modification unless based on reliable, site-specific information.

6-1704.4 (94-06-PFM) Water bodies with perennial flow shall be identified using a scientifically valid system of in-field indicators of perennial flow as determined by the Director. Acceptable methods include but are not limited to the perennial stream mapping protocol developed by the Department of Public Works and Environmental Services and methods determined by the Virginia Department of Conservation and Recreation, Division of Chesapeake Bay Local Assistance to be scientifically valid that are acceptable to the Director.

6-1704.4A (94-06-PFM) Water bodies identified as perennial on the adopted map of Chesapeake Bay Preservation Areas are based on field studies conducted by the Department of Public Works and Environmental Services using established protocols and shall only be reclassified as intermittent based on observations of the absence of stream flow during normal or wetter than normal hydrologic conditions.

6-1704.4B (94-06-PFM) The weekly drought assessment under the U.S. Drought Monitor (NOAA et al) shall be used to determine the general hydrologic conditions at the time of observation. Observations of the absence of stream flow will not be accepted as definitive proof that a stream is intermittent if the weekly U.S. Drought Monitor classification is D0 (abnormally dry) or drier at any time during a period extending from 20 days prior to the date that the first set of observations required by § 6-1704.4D are made through 20 days after the date when the second set of observations required by § 6-1704.4D are made.

6-1704.4C (94-06-PFM) Water bodies not identified as perennial on the adopted map of Chesapeake Bay Preservation Areas may only be reclassified as perennial in conjunction with an amendment to the map by the Board of Supervisors. [2]

6-1704.4D (94-06-PFM) Observations of stream flow shall be made in accordance with the following:

6-1704.4D(1) (94-06-PFM) Unless modified by the Director (e.g. if access to offsite properties is denied or the final upstream limit of the perennial stream lies within the property and is greater than 150 feet (38m) from the downstream property line), observations of stream flow or lack thereof shall be made at intervals of 50 feet (13m) or less along the stream channel beginning a minimum of 150 feet (13m) downstream from the property line to a point a minimum of 150 feet (38m) above the terminus of the perennial stream as depicted on the adopted map of Chesapeake Bay Preservation Areas, at all control sections within the study reach, and at the nearest control section upstream and downstream from the property boundary. A control section is a culvert or other section with a hard bottom where flow would be readily visible.

6-1704.4D(2) (94-06-PFM) Two sets of observations at the above locations must be made a minimum of seven but no longer than thirty days apart.

6-1704.4D(3) (94-06-PFM) Observations shall be made at the true channel bottom which is located below the moveable bed material. Where the channel bed is armored, the presence of flow within the armoring layer must be checked.

6-1704.4D(4) (94-06-PFM) The Department of Public Works and Environmental Services (DPWES) shall be advised prior to or within three days of completion of the first set of observations of the property owner's intent to submit an RPA boundary delineation study to reclassify the stream from perennial to intermittent. DPWES will perform a field review as part of the evaluation of the reclassification study. DPWES will coordinate the field review with the 2nd visit to the site by the agent of the landowner whenever possible. Where there are visible pools of water within the channel that do not appear to be moving, dye tracing and tracing techniques in accordance with ASTM or USGS methods, or other methods and techniques, shall be utilized by DPWES to determine if water is flowing from pool to pool.

6-1704.5 Wetland determinations shall be performed using methods specified by the United States Army Corps of Engineers (USACE).

6-1704.6 RPA boundary delineation studies shall be sealed by a professional engineer, land surveyor, landscape architect, soil scientist, or wetland delineator certified or licensed to practice in the Commonwealth of Virginia. Any work performed by other firms or individuals not under the responsible charge of the licensed professional sealing the study shall be identified and sealed by that individual as appropriate.

6-1704.7 RPA boundary delineation studies shall be submitted on standard-size sheets of 24" X 36" or the metric equivalent at a scale of 1"=50' (1:500) or larger meeting the requirements of § 2-0201.2.

6-1704.8 RPA boundary delineation studies to determine site-specific RPA boundaries shall include the following:

6-1704.8A Cover sheet with project name, County plan identification number, vicinity map, tax map reference, and fee computation;

6-1704.8B A narrative describing how the RPA boundary was established including a discussion of which components listed in § 6-1704.2 determine the RPA boundary and any wetlands shown on the plan that were determined not to be a component of the RPA (i.e., did not meet the requirement of § 6-1704.2D).

6-1704.8C Plan sheet(s) with 2 foot (0.5m) contour interval topography showing each individual component of the RPA overlain to create the final RPA boundary, the RPA boundary from the adopted Chesapeake Bay Preservation Area maps, locations of horizontal and vertical control points, and locations of points and transects used in the wetland determination. Topography shall be correlated to a USGS or County benchmark(s), based on NGVD29, which shall be referenced in the plan. Plan sheets shall include a north arrow in accordance with § 2-0212.3.

6-1704.8D Standard USACE data forms used in the wetland determination and any relevant correspondence from the USACE.

6-1704.8E (94-06-PFM) Source of the major floodplain boundary.

6-1704.9 (94-06-PFM) RPA boundary delineation studies to reclassify streams from perennial to intermittent shall include the following:

6-1704.9A (94-06-PFM) Cover sheet with project name, County plan identification number, vicinity map, tax map reference, and fee computation;

6-1704.9B (94-06-PFM) A narrative describing how, when, and where the observations were made, the weather conditions at the time the observations were made, and the study's final conclusion on whether the stream is perennial or intermittent.

6-1704.9C (94-06-PFM) Plan sheet(s) with 2 foot (0.5m) or 5 foot (1.25m) contour interval topography showing the RPA boundary from the adopted Chesapeake Bay Preservation Area maps, locations of points where observations were made with a key to the photographic documentation provided, the point at which the stream transitions from perennial to intermittent and the revised RPA boundary. Topography shall be correlated to a USGS or County benchmark(s), based on NGVD29, which shall be referenced in the plan. Alternatively, property and topographic information from the County's Geographic Information System may be used. Plan sheets shall include a north arrow in accordance with § 2-0212.3.

6-1704.9D (94-06-PFM) Meteorologic data. Daily precipitation, maximum and minimum temperature, and cloud cover from the nearest National Weather Service weather station for a period of 20 days preceding the date that the first set of observations were made through 20 days after the date when the second set of observations were made. The weekly U.S. Drought Monitor classification for a period of 20 days prior to the date that the first set of observations were made through 20 days after the date when the second set of observations were made. The County may use meteorologic data from local rain gauge stations closer to the site in evaluating the reclassification request.

6-1704.9E (94-06-PFM) Observations of streamflow. The date, time, name of the observer, weather conditions at the time of observation, and photographs looking upstream and downstream documenting each observation. Photographs shall capture the various stream features (e.g. pools, riffles, and runs) along the stream. Photographs of the stream shall be taken close enough to see the channel bed and banks, shall show the channel bottom and any armoring materials, and shall be unobstructed by vegetation. If a clear view cannot be obtained by relocating the point of observation, vegetation may be trimmed to obtain a clear view. Photographs of the channel shall include identifiable stationary landmarks in the field, so that the point of observation can be verified at a later date, if necessary. Identifiable landmarks include survey markers with identification, structural objects such as culverts, bridges, and nearby buildings, or unique natural; features. Photographs must have a visible date stamp or certification by the observer of the date the photographs were taken.

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[1] (94-06-PFM) The values stated in English units are the standard for regulatory purposes. The values given in parentheses are mathematical conversions to metric units that are provided for information only and are not to be considered standard.

[2] (94-06-PFM) Any request to re-evaluate a stream segment for possible reclassification from intermittent to perennial should be made through the Board member in whose district the stream segment is located. The Department of Public Works and Environmental Services will re-evaluate the stream segment and provide a recommendation to the Board member.